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ENGINEERING NEWS-RECORD

DEVOTED TO CIVIL ENGINEERING
AND CONTRACTING

E. J. MEHREN
Editor

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Leninism Rejected

LENINISM, outside of Russia, received three severe setbacks last week. The Italian workmen in the metal industry acquiesced in the government's proposed settlement and began handing back the plants to their owners. In France the Labor Congress definitely repudiated all attempts to control French affairs from the outside, having Moscow particularly in mind, and the American Federation of Labor, through Gompers and Woll, one of the vice-presidents, denounced the efforts of the British labor party to strengthen the Soviet power in Russia. All of which is evidence that the world is getting back to sounder thinking. The danger of the miners' strike in Great Britain, commented upon in these columns last week, also seems to be passed and with its passing goes, for the time at least, another revolutionary proposal. There is reason for confidence and satisfaction, and for trying by sound means to work out the problems between employer and employed.

Cutting Engineering Costs Unwisely

H. K. BISHOP, for sixteen months chief engineer of the Indiana State Highway Commission, was removed from office Sept. 1 because the highway appropriation "had been cut heavily by the tax board." The salary of the chief engineer has been $7,000 a year. Because surveys, plans and specifications covering work for a year in advance had been prepared the director of the commission felt justified in relieving Mr. Bishop. Seven thousand dollars a year is not a large salary for a highly trained technical man of 15 years experience with Federal and state roadbuilding organizations, particularly when expenditures he directs and work he superintends approximate one thousand times his salary. "Saving" the salary of one in such a responsible position is false economy. Competent engineering supervision should and must be paid what it is worth. Smaller salaries than that received by Mr. Bishop for the administration of large funds may mean mediocre engineering.

Moderation and Conciliation

FOUR WEEKS ago we expressed the view that there was need in the American Society of Civil Engineers to drop the bitter spirit which has been injected in discussions since last January and to get down to solid technical work. The response to that editorial expression has been widespread. New evidence has come that the members are tired of strife. They must, perforce, take interest in pending questions until they are settled, but they are anxious to see them disposed of and out of the way. Signs are, too, that those who have been extreme in their statements are inclining toward greater moderation not only of utterance but of view. There were those who, in the heat of discussion, were willing to believe their opponents guilty not only of insincerity but of a desire to use the society for their personal ends. Such beliefs are giving way to a realization that all parties are sincerely interested in the advancement of the society. They differ as to the effects of proposals and the wisdom of means. It is also realized now that of extremists, on either side, there are very few. This attitude is a clear gain. If it lead to moderate discussion, to dispassionate clarification of view on whatever else may need decision before the road is clear for solid technical work, it will help toward speedy and correct decisions.

A Decisive Step

FOLLOWING on years of discussion of projects for bridging the Delaware River at Philadelphia, the appointment of an engineering board to report on location and construction at last brings the enterprise to a definite start. Important as the step is for the immediately adjoining region, it may prove to have equal or greater importance for the country as a whole, by initiating the period of large highway bridge construction which, according to indications, lies just ahead. There has been little activity of this kind for years, and meanwhile the delays and losses resulting from the interruption of the national road system at main rivers have been making themselves more and more painfully felt. At Philadelphia it was the formidable growth of a demand for unhindered communication between the communities on the two banks that brought about definite action on the bridge project. The same demand exists wherever large streams cut the land in two, and it is bound to grow until it brings about the provision of suitable crossings. Thus, much large highway bridge construction is certain to be demanded and carried out in the early future. The Delaware bridge is the first undertaking of the new era, and it should give effective encouragement to many projects that are delayed by mere want of the courage to go ahead.

Improving Waterwheel Efficiency

THE description of the new record-breaking Niagara turbines which appeared in last week's issue is fittingly rounded out by Mr. Horton's résumé of hydraulic turbine practice on another page of this issue. Waterwheel design has progressed rapidly until now the finished machine is a marvel of efficiency and simplicity. In the large units particularly the small number and lack of complication of parts is remarkable and not even approached by any other kind of prime mover. Just now, however, improvements seem to be looked for only in mechanical refinements and Mr. Horton points out that while these are always desirable a more fertile field may possibly lie in hydraulic improvement, that is in some vital re-

arrangement of parts which would more nearly insure a full recovery of the energy of the water. Toward this end he suggests the necessity of minimizing the constraint of the water passing through the unit. In effect this is what many of the later improvements of the turbine do, but it is Mr. Horton's idea that carried further the idea may result in improving the full load-range efficiency, which has not kept pace with the full-gate efficiency. At any rate, while the principle may not go so far it is responsible for certain improvements that help to make the hydraulic turbine the prime mover paramount.

Financing Public Improvements

AN ARRANGEMENT worked out for financing the $200,000 St. Mary's Ave. grading and repaving project in Omaha deserves attention in view of present financial conditions. Seven interested property owners have deposited with a trust company funds sufficient to pay the contractor each month 80 per cent of the engineers' estimate. Upon completion of the work the city will issue district improvement bonds, which the company will carry until the special taxes have been paid. The principal object of the plan was to attract a large number of contractors who could not afford to tie up their capital for so long a period as would be necessary to carry out the work. R. N. Towl, commissioner of public works, who developed the scheme, is of the opinion that the 84-cent bid on earthwork was obtained because the payments were to be made available at once. Much added competition was obtained and the price is 14¾ cents lower than the Dodge St. grading work let a year ago.

Thicker California Roads

BY DECIDING to increase the minimum thickness of concrete pavement from 4 to 5 in. (see News Section, Sept. 30, p. 671) the California Highway Commission moves in a direction that has been frequently urged upon it. In addition, the 5-in. road is to be reinforced. When the 4-in. pavement, with a ½-in. topping, was adopted about ten years ago, there was much criticism of the "shortsightedness" of such a policy. As the roads have aged, this criticism, instead of abating, has ripened into attack of one sort or another. At present it is active in the form of an injunction that has, for the time being, stopped work on every State highway contract. It goes without saying that the commission knew a 4-in. pavement was not the best that could be built. It was decided, however, that the broad way to begin on a highway system that would adequately serve California's great expanse of territory was to plan for the greatest possible mileage of well-located and well-graded road with as good a paving as was consistent, all factors considered. By strict requirements for well-prepared sub-grades, careful inspection of materials and mixing, and under California's favorable climatic conditions, the commission has added to the State road system over seventeen hundred miles of paved highway that has given good service. Evidence of what the people who use these roads think of them, as well as an indication of how this maximum mileage policy (always on a sound technical basis) won friends when they were badly needed, is found in the record of votes cast for the successive bond issues. The first issue ($18,000,000, in 1910) was vigorously opposed in some sections and was carried by the narrow margin of 1.3 to 1. The second ($15,000,000, in 1916) carried by 3.96 to 1, and the latest ($40,000,000, in 1919) by a majority of 7.03 to 1. Meanwhile, pavement has gone to pieces here and there, but on the whole it has stood up well, often under extremely heavy trucking operations that were never thought of until the concrete pavement appeared.

Doubtless the decision for a 5-in. minimum does not mean that thicker pavements will not be built. Surely they are justified in districts where exceedingly heavy trucking operations are carried on, such as in the rice-growing districts of the Sacramento valley. There are those who doubtless will contend that this minimum is not enough under any conditions to be found in the state. Nevertheless, the commission's decision to increase the minimum thickness to 5 in. and to use reinforcing is based on a long experience under one technical head, Austin B. Fletcher, who has been chief engineer of the commission since its formation. Under these circumstances one can safely look upon the new move as being as sound technically as was the adoption of the 4-in thickness nearly ten years ago.

Commercial Aviation Progress

COMMERCIALLY aviation has not made as great progress in this country as in Europe. Here its greatest commercial use is in the mail service, a fact emphasized by the recent institution of the San Francisco-New York service, and the letting of contracts for Pittsburgh-St. Louis and New York-Atlanta routes. Of passenger business, however, there has been very little. The line between New York and Atlantic City, which has maintained regular service for some years, has required only one plane. Some of the eastern stores, for advertising purposes, have delivered rush parcels to their customers by airplane, and at one or two summer resorts flying has been among the season's amusements. Abroad, the great development has been the London-Paris service, but the London-Amsterdam and the Paris-Brussels routes are just as thoroughly established, though the travel by them is not as great. Four lines conduct daily services between London and Paris, a minimum of six planes a day being used in each direction. There have been occasions when as many as twelve machines left London in a single day. As late as last May the cost per flight was £15 15s. (approximately $75 at normal exchange), but the price has now dropped to £10 10s. (about $50). Short-distance flying, also, is more popular in Europe than here. In May while the golf tournaments were on at the Gleneagles course near Glasgow there was regular airplane service to the course from both Glasgow and Edinburgh, services widely advertised in both cities, in a matter-of-fact way. At summer resorts airplanes are popular forms of amusement, in Switzerland, for example, a single company maintaining no less than eighteen machines. Fifteen-minute flights were made at a charge of 90fr. for two persons, or about $9 per person. These planes may be engaged, also, for flights between any given points, tariffs being arranged and published covering a variety of conditions.

The popularity of the airplane can be gathered from the fact that in the year closing June 30 last the correspondent of the Paris edition of the New York *Herald* estimated, from conference with airplane interests, that 30,000 passengers have been carried in

commercial, pleasure, and exhibition flights at the grounds where commercial service is available in England alone. In these flights not a single casualty or serious accident was recorded. The leading English newspapers, notably the London *Times*, published daily a summation of "aviation weather," giving the wind direction and velocity in the upper air at 2,000, 5,000, 10,000, and 15,000-ft. elevations not only for the course between England and the northern continental points of Amsterdam, Brussels and Paris, but also for the whole of England and Scotland as well.

Lloyd's, the famous insurance house, as is well known, has for some years been writing insurance both upon passenger and goods transportation. So important has this branch of the business become, however, that it has recently begun the publication of an *Aviation Record* to serve the same purpose as its famous *Shipping Record*. It will contain complete listings of the airplanes and airplane companies furnishing commercial service, with records of the pilots, the histories of all the machines, etc., so that there can be, as time passes, more intelligent assessment of rates. In addition, Lloyd's has set up a technical committee on the order of its famous shipping committees, on which are represented the aircraft transport companies, aircraft builders, and underwriters. The purpose is to make such studies and recommendations as will decrease the risk of transportation by aircraft. It is expected that, in time, this will lead to certain standards for commercial airplanes.

While, therefore, through our mail service commercial transportation by airplane is making some progress in America we are far behind Europe. The progress there bears out the predictions which were made as many as ten years ago that the airplane would most certainly become a factor in commercial transportation.

Passing the Crisis in Transportation

WITH the end of maximum coal movement on the railroads this fall there is reason to look forward to at least temporary relief from the transportation crisis through which the country is passing. As a result of the ambitious program of the Association of Railway Executives to increase tonnage movement, most encouraging results have been secured by the efforts of the railroads, with the co-operation of shippers, as shown in current reports from almost all of the larger carriers. A very material increase in carrying capacity has been obtained with no increase of plant. To the construction industry the prospect should appear decidedly encouraging, but there will by no means be satisfaction with a situation so far from perfect. The construction industry itself, through the large proportion of shipping which it represents, can go far toward effecting immediate relief by the fullest co-operation with the railroads in loading and release of cars. But while temporary relief may be had by emergency measures—which even have included much Sunday work—we believe that the permanent cure of transportation ills lies in the hands of the engineer.

Constructive suggestions for broadening the engineer's functions in railroad operation may be found in the article on p. 693, in which the author brings out sound reasons for the proposal that the engineer be given definite recognition by creating the office of "transportation engineer." As the close relation of the engineer to railroad operation is seen more clearly by the managers, this new office would seem to be a logical development and, in fact, a move in this direction has been made by at least one large system. Through his training and exact knowledge the engineer is peculiarly well fitted to the duties proposed, which would open a new field not limited to design and construction.

For the reason that "transportation problems will be solved only by the application of engineering knowledge," it is proposed that every large railway system appoint an engineer to study problems of transportation, vesting him with the necessary authority. It would be his duty to apply engineering methods and logic to many phases of operation which have always been ruled largely by tradition—such as switching and freight-handling costs in large terminals. There is growing belief that the handling and rehandling of cars at many terminal points could be materially reduced and a decided improvement effected in the traditional methods generally used for handling less-than-carload-lots and trap car freight in most of the cities. Here, indeed, lies a vast field of opportunity for the application of engineering principles to such phases of operation as have heretofore been largely governed by custom and by rule of thumb.

Since so long a time must be consumed in extending and revising present transportation facilities to bring them to a point of adequate traffic carrying capacity, emergency methods must be used for some time. We believe that the railroad managements—realizing that failure to provide service would result in government operation, if not in government ownership—have made a supreme effort to meet the emergency. The results of the 30-ton-30-mile-per-day program of the railway executives merit the highest degree of co-operation on the part of all shippers. By capacity loading, regardless of low tariff minimums, and by quick unloading of cars, regardless of demurrage allowance, still more satisfactory results will be obtained.

Recently joint car service committees, consisting of construction men and railway executives, have been established in Boston, New York, Philadelphia, Baltimore, Pittsburgh, Cleveland, and Chicago to secure cars and materials needed for necessary construction which is actually under way and which is delayed or entirely stopped by lack of materials. The railroad representatives were appointed by the Association of Railway Executives, and the construction representatives were nominated by the National Federation of Construction Industries and appointed by the United States Senate's Special Committee on Reconstruction. Appeals for necessary car service should be addressed to the joint committees and if cars are not produced appeal may be made to the chairman of the Advisory Committee of the Association of Railway Executives, and finally to the Interstate Commerce Commission, but so far as is known all requests have been cared for by the joint committees without appeal.

A basis for effective co-operation has been formed between the railroads and the construction industry. In general, the railroads have demonstrated their good faith by the gratifying results already obtained. In the interest of the whole community, as well as the parties immediately concerned, the construction industry should and will do no less than its part in aiding the railroads by full loading, prompt unloading and the most effective use of cars.

Analysis of the Continuous Three-Column Foundation

Distribution of Base Pressure Controlled by Continuity of Main Grillage Girders—How Correct Lengths of Girder Projections Are Computed—Special Case of Limited Projection

By Charles A. Ellis
Professor of Structural Engineering, University of Illinois, Urbana

IN DESIGNING foundations for high buildings the engineer frequently must place three columns on a single footing, using longitudinal supporting girders resting on shorter cross-beams bearing on the soil, in order to distribute the column loads over a sufficient area of foundation. In such a foundation he endeavors to obtain uniform soil pressure for the support of the concentrated columns loads involved He fixes upon the allowable unit bearing pressure of the soil, determines therefrom the required foundation area, and then shapes the footing by assuming arbitrarily either the length or the width of the footing to suit his convenience The last step of the process disregards the fact that the girders and their loading constitute a statically indeterminate system. Arbitrary proportioning of such a system results in making the soil pressure non-uniform (hence of excessive intensity at some points) and the bending moments in the foundation girders different from those contemplated in his design, which means reduced safety and economy.

A rational analysis of the problem is quite feasible. As the subject does not appear to have been discussed

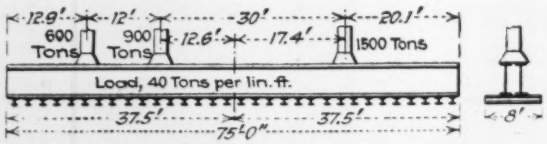


FIG. 1. TYPICAL THREE-COLUMN GRILLAGE FOUNDATION AS PROPORTIONED BY CUSTOMARY METHOD

previously, the writer proposes to give the theory and outline its application to several practical cases. From the theoretical standpoint there is a remaining difficulty in the solution presented: Since the foundation girders distort slightly in performing their office of distributing the loads, the assumption of uniformly distributed soil bearing pressure which the designer necessarily makes (in the present analysis as well as in customary procedure) is unavoidably vitiated. We have no data as to the importance of this effect, but it seems reasonable to conclude that a state of uniform pressure distribution is more nearly approximated when the problem is solved by an analysis that takes account of the continuity of girders than by the present arbitrary choice of proportions.

How the customary method involves arbitrary proportioning may be illustrated by reference to Fig. 1. In the case there sketched, the three column loads total 3,000 tons. A permissible soil reaction of 5 tons per square foot is assumed; hence the required footing area is 600 sq.ft. Next, the length and width of the footing are arbitrarily fixed by the designer as 75 ft. and 8 ft. respectively. The designer then determines the left-hand and right-hand projections of the main girders by finding the center of gravity of the three column loads (12.6 ft. to the right of the center column), and making this the center of gravity of the soil reaction, which requires that the foundation should extend 12.9 ft. to the left of the left column, and 20.1 ft. to the right of the right column.

A grillage so designed satisfies the three conditions of static equilibrium (sum of vertical forces, sum of horizontal forces, and sum of moments about any point, respectively, equal zero). It is therefrom concluded that the three concentrated loads are supported by a

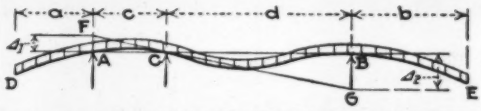


FIG. 2. DIAGRAM OF DEFLECTION OF UNIFORMLY LOADED BEAM ON THREE SUPPORTS (Inversion of Three-Column Foundation)

uniform soil reaction of 40 tons per lineal foot. That this conclusion is fallacious, however, becomes apparent when the structure in Fig. 1 is inverted, and the soil pressure represented as a uniform load of 40 tons per lineal foot resting upon three supports, whereupon it is seen at once that the problem is statically indeterminate and that the odds are greatly against the probability of the reactions amounting to just 600, 900 and 1,500 tons respectively. In other words, the column loads being the given quantities, the soil pressures cannot be uniform, as assumed, and therefore it is likely that the bending moments in the girders will differ materially from those anticipated by the designer.

Girder on Three Supports

By way of introduction to the rational method, the general case of a girder resting on three supports and uniformly loaded (Fig. 2.) will be considered. Three independent simultaneous equations are required for the determination of the three reactions. The equation $\Sigma H = 0$ lends no aid to the solution, since all the forces act vertically. The two remaining static equations, $\Sigma V = 0$ and $\Sigma M = 0$, are necessary but not sufficient. The problem is therefore statically indeterminate. The third equation necessary for a solution is based upon the elastic theory of beams, and may be derived from the assumption that the three points of support A, C and B remain in a straight line (not necessarily a level line).

Let FG represent the tangent to the elastic curve $DACBE$ at C, and let $AF = \Delta_1$ and $BG = \Delta_2$ represent the tangential deviations of this curve at A and B respectively. Since these deviations are on opposite sides of the tangent, $\Delta_1/\Delta_2 = -c/d$, which is the third equation required.

Application of the three equations to practical cases differs in detail according to the physical conditions governing the problem. Three of the several possible cases are illustrated in Figs. 3, 4 and 5, and will be considered separately. In each case the following points are to be observed:

Each figure is shown inverted, for convenience. The known column reactions are represented by P, Q and R. The spacings of the columns, being fixed by the architectural features, are known, and are represented by c and d. The tangential deviations Δ_1 and Δ_2, not shown in the figures which follow, are to be taken as represented in Fig. 2. Three quantities, differing in each case, are to be determined by a solution of the three independent simultaneous equations cited.

Case I—Projections Not Limited by Site

Referring to Fig. 3, let w represent the intensity of a uniform soil pressure, in pounds per lineal foot. If the architectural conditions do not limit the end projections a and b of the main grillage girders, then it will be possible to attain this condition of uniform pressure by selecting a, b and w accordingly.

Equilibrium of vertical forces and of moments (in this case taken about the left end) gives two equations,

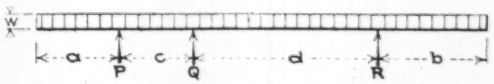


FIG. 3. DIAGRAM FOR ANALYSIS OF THREE-COLUMN FOUNDATION

the third being that for tangential deviation, derived above:

$$P + Q + R - (a + b + c + d)w = 0 \qquad (1)$$

$$aP + (a + c)Q + (a + c + d)R - \frac{w}{2}(a + b + c + d)^2 = 0 \qquad (2)$$

and

$$\frac{\Delta_1}{\Delta_2} = -\frac{c}{d} \qquad (3)$$

The expressions for Δ_1 and Δ_2 may be determined as follows: The bending moment between P and Q at any distance x from P is $M_1 = Px - \frac{1}{2}w(a + x)^2$, and the tangential deviation at P therefore becomes,

$$\Delta_1 = \frac{1}{EI}\int_0 M_1 x\,dx = \frac{1}{EI}\int_0 \left[Px^2 - \frac{wx}{2}(a + x)^2\right]dx$$

$$= \frac{1}{EI}\left[\frac{c^3 P}{3} - \frac{w}{2}\left(\frac{a^2c^2}{2} + \frac{2ac^3}{3} + \frac{c^4}{4}\right)\right]$$

Similarly, the bending moment between Q and R at any distance x from R is $M_2 = Rx - \frac{1}{2}w(b + x)^2$, and the tangential deviation at R is, therefore,

$$\Delta_2 = \frac{1}{EI}\int_0 M_2 x\,dx = \frac{1}{EI}\int_0^d \left[Rx^2 - \frac{wx}{2}(b + x)^2\right]dx$$

$$= \frac{1}{EI}\left[\frac{d^3 R}{3} - \frac{w}{2}\left(\frac{b^2d^2}{2} + \frac{2bd^3}{3} + \frac{d^4}{4}\right)\right]$$

Inserting these values in eq. (3),

$$\frac{8c^3P - w(6a^2c^2 + 8ac^3 + 3c^4)}{8d^3R - w(6b^2d^2 + 8bd^3 + 3d^4)} = -\frac{c}{d} \qquad (3a)$$

The unknown quantities a, b and w are to be determined by solving simultaneously the three independent equations (1), (2) and (3a), as in the following numerical problem.

Illustrative Problem—Let $c = 12$ ft., $d = 30$ ft., $P = 600$ tons, $Q = 900$ tons and $R = 1{,}500$ tons. To solve for a, b and w, eliminate w from (1) and (2), and from (1) and (3a), obtaining,

$$b = a + \frac{c(-P + Q + R) + d(-P - Q + R)}{P + Q + R}$$

or, $b = a + 7.2$

and

$$6c(P + Q + R)a^2 + 6d(P + Q + R)b^2 + 8(c^2Q + c^2R - d^2R)a + 8(d^2P + d^2Q - c^2P)b + (-5c^2 + 3d^2 - 8c^2d)P + 3(c^2 + d^2)Q + (-5d^2 + 3c^2 - 8cd^2)R = 0$$

or $$10a^2 + 25b^2 - 372a + 468b = 10374$$

whence,

$$a = 7.8 \text{ ft.}, \quad b = 15 \text{ ft.}$$

The length of the base, then, is 42 + 22.8 ft. or 64.8 ft., and the soil pressure per lineal foot is $w = 3000/64.8 = 46.3$ tons. If the permissible bearing pressure on the soil is 5 tons per square foot, the foundation should have a width of $46.3/5 = 9.5$ ft.

Thus in Figs. 1 or 3, the girders should be about 65 feet long, extending approximately 8 ft. beyond the left column and 15 ft. beyond the right column. The beams of the lower tier of grillage should have a length of about 9.5 feet.

Case II—Projection at One End Limited by Site

Architectural features frequently fix the length at which the footing may extend beyond one of the end columns. Sometimes it is necessary to allow no extension whatever at one end. When either of these two limitations arises the footing may be so arranged that its pressure on the soil per foot of length varies

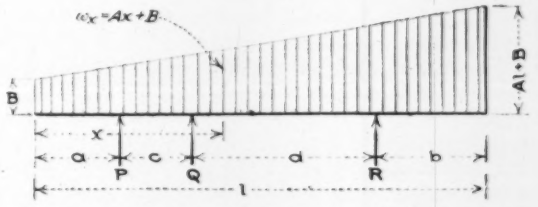


FIG. 4. DISTRIBUTION OF BASE PRESSURE WHEN PROJECTION AT ONE END IS FIXED

uniformly from one end to the other, as shown in Fig. 4. This may be accomplished by a variation in the lengths of the cross-beams, resulting in a trapezoidal area for the footing. Care should be exercised in the choice and spacing of the cross-beams in order that equal deflections at their centers may be assured.

Let the soil pressure per lineal foot at a distance x from the left end be,

$$w_x = Ax + B$$

Then the soil pressure at the left end will have the intensity B per lineal foot, and at the right end the intensity $Al + B$ per lineal foot. The three equations then become,

$$\Sigma V = Al^2 + 2Bl - 2(P + Q + R) = 0$$

$$\Sigma M = 2Al^3 + 3Bl^2 - 6aP - 6(a + c)Q - 6(a + c + d)R = 0$$

and $$\frac{\Delta_1}{\Delta_2} = -\frac{c}{d}$$

The latter, on computing the values of Δ_1 and Δ_2, gives,

$$c\Big[4(a+c)^3+a\Big\{3(a+c)^2+a(3a+2c)\Big\}\Big]A$$
$$+d\Big[(a+c+d)^4$$
$$+(a+c)\Big\{2(a+c+d)^3+(a+c)\Big[7(a+c)+3d\Big]\Big\}\Big]A$$
$$+5c\Big[3(a+c)^2+a(3a+2c)\Big]B$$
$$+5d\Big[(a+c+d)^2+(a+c)\Big\{5(a+c)+2d\Big\}\Big]B$$
$$=20\Big[(c+d)^2+c(c+d)\Big]P+20d^2Q$$

Three of the four quantities a, b, A and B may be determined from these equations. Either a or b is limited by the architectural conditions as we have assumed, and therefore the equations suffice for complete solution.

Illustrative Problem—Let $P = 600$ tons, $Q = 900$ tons, $R = 1{,}500$ tons, $c = 12$ ft. and $d = 30$ ft. The conditions of the site do not allow a to exceed 6 ft., and the designer desires to utilize this full amount; in other words, he fixes a at 6 ft., and has to determine A, B and b from the equations. These latter become,

$$Al^2 + 2Bl = 6000$$
$$2Al^2 + 3Bl^2 = 550800$$

and, $$8273664A + 824040B = 43416000$$

whence $$l^3 - 227.761l^2 + 13884.448l - 209928.788 = 0$$

$$l = 22.78 \text{ ft.}$$

or, $$l = 66.58 \text{ ft.}$$

$$\text{or } 138.4 \text{ ft.}$$

If 66.58 feet is taken, then $B = 55.98$ and $A = -0.328$; the intensity at the right end is $Al + B = 34.13$. That is, the reaction has an intensity of 56 tons per lineal foot at its left end, and decreases uniformly to 34 tons per lineal foot at its right end. Since $a = 6$ and $l = 66.6$ ft., the foundation extends $b = 18.6$ ft. beyond the right column.

The equations given for this case may be applied just as readily when the foundation may not extend beyond one of the outer columns (as when $a = 0$).

Case III—Both Projections Limited by Site

In the case illustrated by Fig. 5, where both a and b either are fixed or equal zero, the footing may be so proportioned as to produce a soil reaction whose intensity per lineal foot varies according to a parabolic curve. The soil pressure per square foot may be made uniform by varying the length of the lower beams of the grillage accordingly.

The dimensions a, b, c and d, and the loads P, Q and R will be known, while the three constants which determine the intensity of the soil reaction are to be determined by the solution.

Let the intensity of the soil reaction per foot of girder be

$$w_x = Ax^2 + Bx + C$$

so that at the left end the intensity will be C and at the right end $Al^2 + Bl + C$ per lineal foot. The three equations then become,

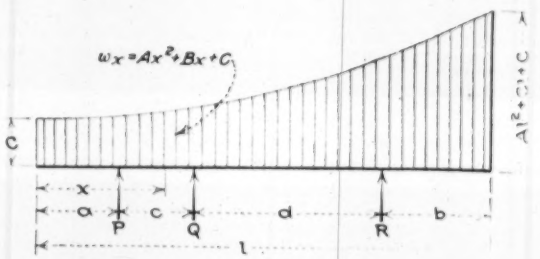


FIG. 5. BOTH PROJECTIONS LIMITED BY CONDITIONS AT SITE

$$\Sigma V = 2Al^3 + 3Bl^2 + 6Cl - 6(P + Q + R) = 0$$
$$\Sigma M = 3Al^4 + 4Bl^3 + 6Cl^2 - 12aP - 12(a + c)Q - 12(a + c + d)R = 0$$

and from $\Delta_1/\Delta_2 = -c/d$, the third equation,

$$c\Big[5(a+c)^4+a\Big\{4(a+c)^3+a\Big[3(a+c)^2+a(3a+2c)\Big]\Big\}\Big]A$$
$$+d\Big[(a+c+d)^4+(a+c)\Big\{2(a+c+d)^3$$
$$+(a+c)\Big[3(a+c+d)^2+(a+c)\Big\{9(a+c)+4d\Big\}\Big]\Big\}\Big]A$$
$$+3c\Big[4(a+c)^3+a\Big\{3(a+c)^2+a(3a+2c)\Big\}\Big]B$$
$$+3d\Big[(a+c+d)^3$$
$$+(a+c)\Big\{2(a+c+d)^2+(a+c)\Big[7(a+c)+3d\Big]\Big\}\Big]B$$
$$+\Big[15c\Big\{3(a+c)^2+a(3a+2c)\Big\}$$
$$+15d\Big\{(a+c+d)^2+(a+c)\Big[5(a+c)+2d\Big]\Big\}\Big]C$$
$$=60\Big[(c+d)^2+c(c+d)\Big]P+60d^2Q$$

Illustrative Problem—Let $P = 600$ tons, $Q = 900$ tons, $R = 1{,}500$ tons, $a = 6$ ft., $b = 12$ ft., $c = 12$ ft., and $d = 30$ ft. Then $l = 60$ ft.

Substituting the numerical values in the above equations and reducing,

$$1200A + 30B + C = 50$$
$$1800A + 40B + C = 51$$
$$623028A + 38304B + 3815C = 201000$$

The solution of these equations gives $A = 0.0291$, $B = -1.646$, and $C = 64.466$, and the intensity of the reaction at any point is, therefore, $w_x = 0.0291x^2 - 1.646x + 64.466$. Values of the reaction computed for the ends and five intermediate points are:

x Ft.	w Tons / Lin. Ft.	x Ft.	w Tons / Lin. Ft.
0	64.5	40	45
10	51	50	55
20	43	60	70.5
30	41		

Where the foundation must not extend beyond one of the outer columns, so that, say, $a = 0$, the equations still apply, and substitution of the numerical values in the algebraic forms given for Case III will supply the correct numerical solution. The same is true when the foundation is not to extend beyond either outside column, in which case $a = 0$, $b = 0$ are to be substituted.

Modern Developments of Hydraulic Turbine Design

Precedent and Principle as Exemplified in the New Niagara Wheels—Minimum Constraint a Factor in Efficiency

By Robert E. Horton
Consulting Hydraulic Engineer, Voorheesville, N. Y.

In "Engineering News-Record," Sept. 27, 1920, p. 646, two of the foremost American turbine designers, Lewis F. Moody and W. M. White, described in some detail the new 37,500-hp. turbine units built by their respective companies for the latest American Niagara Falls development. The contrast between these two largest waterwheels yet built and the early foreign-designed and foreign-built wheels installed in American power plants is so marked that this seems a proper time for a résumé of hydraulic turbine practice, with particular relation to American contributions to such practice and the prospects for future improvements. Such an article has been prepared for "Engineering News-Record" by Mr. Horton.—Editor.

WHEN American manufacturers undertook the development of modern specially designed turbines they nearly all imported Swiss or German designers. Without any intended disrespect, it is not too much to say that some, at least, of the foreign designers brought to this country were a bit "cocky" regarding their capabilities. Time has shown that American designers were able to learn all that the Europeans knew, and so far as printed records go there is nothing to show any work in the way of turbine designing done in continental Europe comparable in efficiency, capacity, or flexibility with recent American practice.

Today the hydraulic turbine is the simplest, most reliable, most durable, most efficient and most powerful per single unit of all prime movers. How this has come about is a long but interesting story, which will not be recounted here further than is necessary to show the bearing of precedents on the development of turbine practice and efficiency.

This process has taken place through the slow and costly operation of the logic of exclusion and the "survival of the fittest," but it has taken place almost entirely within the United States. Less than a century ago a turbine having a capacity of one-tenth of one per cent of that of the new Niagara units was about the available limit. Fifty years ago a turbine runner of one-tenth of the capacity of the Niagara turbines was considered large, and it is only recently that single runners as large as 10 per cent of the capacity of the Niagara units have been used. Among the earliest of these were the old Leffel Niagara turbines, deriving water from the same canal which supplies the latest installation.

Features of Niagara Turbines

Features of the new Niagara turbines of special interest are of two classes: (1) Structural and mechanical features, relating to provisions for durability, repair, and convenience of operation. (2) Hydraulic features, relating particularly to considerations of efficiency.

Among the structural features worthy of note are the use of the Johnson penstock valves, renewable parts for wearing surfaces, adjustable gate actuating mechanism, with provision to take up wear and avoid lost motion, complete separate hand control to permit of repairs of governors during operation and the provision of brakes.

In this connection the use of metal volute casings imbedded in concrete calls attention to the fact that although concrete alone has often been used for turbine spiral settings, yet in this instance because of the relatively great head, and the desire to secure maximum efficiency a metal lining of the volute was deemed necessary. One builder has chosen cast iron and the other steel plate for this lining. Arguments in favor of each are given in the articles referred to.

One reason advanced for the use of steel plate volute casings is freedom from casting strains. Still the runner of the same unit is made of cast iron, cast in one piece, and it would seem that in so complicated a casting as a waterwheel runner the danger of shrinkage strains and cold shuts is much greater than in the simpler cast segments of the volute casing.

Features of the design relating especially to hydraulic conditions include the use of carefully designed and formed volutes to supply the water to each wheel—the use of the hydraucone on one and the "spreading draft tube" on the other unit, the use of fixed auxiliary guides, forming also supports for the volute casting, in addition to the use of ordinary efficiency type guide-gates in the I. P. Morris unit, and the use of labyrinth clearance cells to reduce runner leakage from the same unit.

Simplicity Is Remarkable

Preliminary tests of the new Niagara turbines indicate turbine efficiencies of 93 per cent at full load. Part of the remaining 7 per cent is unavoidably consumed in mechanical and bearing friction, and some in skin friction. These losses represent energy supplied from the water, so that considerably more than 93 per cent of the potential energy of the water is abstracted therefrom in passing through the turbine runner. Equally high efficiencies have heretofore been obtained in a few instances, and there is nothing extraordinary about the efficiencies of these turbines, except possibly that the efficiency can be maintained high in so large a machine. The remarkable thing, and one which these particular turbines emphasize, is the fact that any hydraulic turbine can abstract anything like so great a portion of the energy of a stream of water by merely letting the water flow through so simple a thing as a turbine runner. The runner has been called a "thing" purposely, and not a mechanism, because a turbine contains only one moving part, properly speaking; namely, the runner, which in these instances is a single casting, yet capable of deriving 37,500 hp. from a stream of water.

The reader can readily make for himself a comparison of water turbines with other prime movers, as regards simplicity and efficiency. While a hydroelectric power unit, with its turbine, casing, gates, inlet valves, draft tube, and governor, is a fairly elaborate system of mechanism, yet the runner, the part that really does the work, is so simple that one stands in wonderment as to the manner in which so efficient a result is accomplished in such a simple way.

There is an old fundamental tenet of turbine design that "the water must enter without shock and leave

without velocity." Early experimenters naturally assumed that to attain this requirement the water must be guided smoothly and accurately through the turbine along predetermined lines of flow. The prevailing idea seemed to have been that a definite passage must be fixed in advance for each separate filament of water. Practically this could not be attained, but looking toward this end guide and runner vanes were multiplied and contorted. Fins were added in some cases to the vanes. The clearance between the guides and runner was reduced to a minimum, and in some instances grooves and convolutions were cast in the runner vanes, as if with the idea of suggesting to the water the direction it was desired that it should pursue in its course through the wheel.

Minimum Constraint Desirable

The invariable verdict of early Holyoke tests of such complicated contrivances was "weighed and found wanting." The more the water was tortured in its course through the wheel the less power it yielded. It is only recently that the idea of giving the water direction with only a minimum of constraint has been developed. Some direction is necessary as the water enters a turbine runner, as everyone knows water can only enter the buckets without shock when it approaches the wheel with a tangential component of velocity equal to the peripheral velocity of the runner.

The question may naturally be asked—Why did it take nearly a century for turbine inventors and designers to discover that they had hitherto mostly been working along lines contrary to high efficiency? In answering this question it may be stated that there is probably some misapprehension among those not conversant with this highly specialized subject as to just what is meant by turbine designing. Theory, meaning thereby the more formal and mathematical treatment of the subject, furnishes a basis for determining certain elements of a turbine design for specific conditions, including the diameter of the runner, the net cross-section of the water passages, and the guide and bucket vane inlet and outlet angles but theory in the sense referred to does not determine the best number of guides or buckets or the permissible clearance space between the inner ends of the guides and the outer edges of the bucket vanes—nor does it determine either the course or form of cross-section of the bucket passages. The design, so far as these elements are concerned, is a matter of common sense, experience and judgment, combined with certain requirements for strength and rigidity.

Water is a nearly incompressible and relatively frictionless fluid. If the mean path of the filaments of water through a turbine is correctly fixed, then within certain limits each filament forms in effect a guiding surface for the adjacent filaments. In other words, the water itself affords constraint, to which the water flowing through the runner naturally adjusts itself, and which is superior in every way to any fixed material constraint which can be devised. This principle of hydraulics received application in various ways other than in turbine design long before its import was fully appreciated—in fact it seems thus far to have been expressed only more or less vaguely, and never in succinct concrete form.

The matter of reducing the constraint of water in passing through a turbine has been carried to an extreme in a type of waterwheel recently placed on the mar... In this turbine the runner vanes are reduced to th... or four in number—the rim band is omitted—and t... vanes are cut down to little more than a vestige ... their categorical type, so that the runner resembles th... form of a boat propeller wheel. A case and guides ar... provided like those of an ordinary turbine. Such ... construction is reported to give efficiencies well up i... the scale, though several per cent below the maximu... of the best of the present existing types.

Here the reduction of constraint has apparently bee... carried too far for maximum efficiency, but something else has been accomplished, since it is found that such a turbine gives a flatter efficiency curve than any obtained hitherto by designs of the more usual type. Precedents are found for this novel construction, not only in the well-known screw propeller but in some early turbines, such as the Bastian, a waterwheel largely used in the north for sawmills in early days because of its speed and freedom from obstruction. The runner was of the Jonval type, with only two vanes, set at a low angle, and with very large clear openings between them, affording so little constraint to the water that it was said of these wheels that a sawlog could pass through one of the buckets without injuring either the runner or the log.

Possible Improvements

The significance of all this is that it suggests the only possibility remaining open for any material hydraulic improvement in turbine design. Given 93 per cent maximum efficiency, as in the new Niagara turbines, and others of recent construction, there is obviously little to be gained in maximum efficiency, but that figure is attained at virtually one speed and one load only. Over the ordinary range of loads, say one-half to full gate, when running at constant speed, an average efficiency as high as 80 per cent has rarely been attained. It is possible that by a happy combination of limited restraints in guides and runner a turbine may yet be produced that will give an average efficiency near 90 per cent over the full ordinary range of loads, but the prospects are not very bright.

Referring to the matter of constraint of water, one of the significant differences between the Allis-Chalmers and I. P. Morris units of the new Niagara development is the use of preliminary or directive guides in the volute case in conjunction with the latter. Whatever may be their structural utility, some question arises in the light of experience as to their hydraulic value. The use of preliminary guides has abundant precedent, but the result is mostly against their use. There have been advocates of the other extreme, who favor the omission of guides altogether, a practice commonly followed in the early central discharge turbines with volute or scroll cases, and even with some earlier Jonval wheels, and following still earlier precedents of wooden flat vane central discharge wheels, which rarely or never had guides.

In view of the poor construction and meager experimental data of such early waterwheel types, it is impossible to determine with certainty whether the better efficiencies obtained following the insertion of guide vanes in the runner cases were really due to the use of guides or to improved construction. The general belief is that guides are beneficial, if not absolutely necessary, in all turbines, to secure maximum efficiency

at full load and gate, but this belief may rest too largely on their more apparent necessity for wheels in rectangular flumes. For wheels like the new Niagara turbines, in volute cases, there is still a question as to the hydraulic benefit derived from guides under full load conditions, at least. Of course with present-day methods of speed control they cannot be dispensed with since they are integral with the speed gates, but it is worth mentioning that some twenty-three years ago the late J. P. Frizell published an able discussion of water turbine possibilities, in which he suggested that maximum hydraulic efficiency and flexibility might yet be attained through the use of a turbine without guides.

Vortex Energy Recovered

Much of the energy of the water leaving a turbine runner is energy of internal motion in eddies and vortices. If this motion is suddenly arrested the energy will be dissipated, and if the eddies leave the draft tube with the outflowing water the energy is lost. The hydraucone and spreading draft tubes mentioned in the articles on the Niagara turbines appear to recover much of this vortex energy by reconverting it into potential energy within the draft tube itself. This is apparently accomplished through the application of the principle of minimum constraint. The water is constrained only so much as necessary to permit it to follow freely its natural lines of flow, leaving the eddies and vortices to cavort and contort at will and unhindered, until they are slowly and smoothly dispersed by fluid friction. Here, as De Vilamil pointed out a few years ago, internal friction of water may be a positive benefit in increasing the available efficiency, since, as Helmholtz showed many years ago, vortex motion would persist forever in a frictionless fluid, and the energy of eddy motion could not then be recovered. Incidentally, the Boyden diffuser applied about 1850 to early Fourneyron or outward flow turbines appears to have been a precedent of the more modern devices for recovering the energy of discharge from a turbine.

So far as the runners of the Niagara turbines are concerned, they apparently involve nothing new in hydraulics. The form of buckets or water passages through these latest and largest turbine runners differs but little from that developed by John B. McCormick, following the earlier attempts at the production of an efficient inward flow turbine runner by Howd, Francis and Swain.

McCormick's improvements, which have been so widely applied in American turbines of the stock pattern type, consisted in lengthening the bucket passages so as to give the water a chance to escape with a minimum velocity relative to the earth, and the shortening and tapering of the inlet cone, increasing the clearance between the guides and runner vanes. This type still persists, and undoubtedly will persist, and the modern turbine designer only modifies it to the extent of specifying certain dimensions, so that it may be adapted in advance to the specific conditions of load and speed. Incidentally, Mr. McCormick relates that the familiar extended spoon or ladle bucket form was discovered by accident. Under Mr. McCormick's direction, iron plate extensions were riveted on to the bucket outlet edges of an old inward flow turbine at a small Pennsylvania sawmill, to reduce the discharge capacity so that the wheel could run with the water supply available. To his surprise, the modification not only reduced the discharge of water but it also increased the power.

Naturally, other features than efficiency are important in turbine design. This fact is emphasized by the rapid displacement of efficient overshot waterwheels by crude turbines in the early decades of the last century. Overshot wheels, tested by John Smeaton, between 1750 and 1760 in England, gave an efficiency often above 80 per cent, yet such wheels were rapidly replaced by crude turbines having efficiencies often of only 20 to 35 per cent. This was largely because of the high speed and compactness of the turbine wheel.

Spiral or volute casings for turbines have been long in use, dating back at least to the primitive flat vane wooden central discharge wheels of 100 years ago or more—so there are ancient precedents for nearly all the hydraulic features of the most modern turbines. It is in the adaptation of principles and practice formerly applied on a small scale to these large units, and the combination of the good features evolved during the past 100 years, but rarely hitherto all combined in a single turbine unit, which make the latest hydraulic turbine developments so remarkable for efficiency and other characteristics, as compared with the best of other types of prime movers.

There are, however, various matters related to the structural design of powerful turbine units such as these which have to be given very careful consideration, although they may be relatively unimportant in the design of the ordinary smaller hydraulic turbines. These include, for example, the problem of bearings, and the problem of designing a governor mechanism sufficiently sensitive to give speed regulation of a high order, and at the same time sufficiently powerful to move and control the enormous weights and inertia involved.

Advanced Hydraulics

At the recent Niagara professional meeting of the Engineering Institute of Canada, Frank Barber, the Toronto bridge engineer, told this story. He was having dinner at a small hotel in an upper Ontario town with several other transients, among whom was a talkative citizen who might have been a prosperous merchant or banker—at least one accustomed to be listened to. "Nice town you have here" opened this gentleman. "We think so," allowed Barber, not bothering to disavow a local status. "But you ought to develop all of this water power that's running away" said the man. "Well," returned Barber, "It would cost a lot of money." "Not at all," said the visitor, "Not at all. I guess you don't know about the way they develop power these days." "No," inquired Barber, "I guess I don't. Is it cheaper than they used to get it?" "Simple as can be," said the man. "All you have to do is to dig a deep hole in the ground, as deep as you want to provide machinery for, and put your wheels at the bottom of the shaft. You dig a canal to the top of the hole, the water drops down the hole and drives the wheel and there you are." "That's great," said Barber, "But I didn't quite get all the details of the scheme. What did you say became of the water after it got by the wheel?" "Young man" said the authority pityingly, looking around the table with the superior air of one who states the obvious "Young man, I guess you needn't care what becomes of the water after it has got the wheel to moving."

Jackscrews Tighten Wood Block Pavement in Place

Blocks Slid Together Over Pitch Bed Distances of 125 Ft. Each Way From Jacks—Movement in 250 Ft. Reaches 30 In.—Blocks Move Most in Direction Against Traffic

By W. W. Horner
Chief Engineer, Sewers and Paving, St. Louis, Mo.

FACED with the probability of having to take up 7,500 sq.yd. of loose wood block at an estimated cost of about $10,000 there has been developed this year, in St. Louis, a successful method of tightening pavement without removing and relaying the block. Briefly the task was accomplished by cutting into the pavement at intervals, and tightening up the block on each side of the cut with jackscrews. In consultation with John Richards, block paving superintendent, the apparatus shown in Fig. 1 was developed. It consists of 4 x 6-in. timbers laid against the wood block pavement, and practically flush with the pavement surface. One of the timbers bored out takes the screws of short trench braces. These braces are equipped with large hexagonal screw nuts working against steel bearing plates, which rest loosely against the side of the timber. The width of the pavement from curb to rail is about 22 ft., and the jack frames were made 11 ft. long in order more nearly to fit the curve of the crown. Three jacks were used for each set. This original apparatus, and two other sets, slightly modified, were used, under the supervision of Mr. Richards, throughout the summer on this work. It was found possible, with the proper combination of hot weather and pressure on the jackscrews, to slide the blocks on the pitch bed over considerable distances.

Development of Conditions Requiring Repairs

In an article in *Engineering News-Record*, April 22, 1920, p. 814, the writer recited difficulties encountered during the construction season of 1919 with the laying of new wood block pavements in St. Louis. Three projects were described, in the first of which, Contract *A*, the blocks shrank so badly after laying that the pitch filler settled under the blocks, completely ruining the paving. In this instance it was necessary to take up all of the blocks, clean them in tar kettles in creosote oil, re-lay them, and apply new filler. The total cost of this work was about $1.25 per square yard. In the second project, called Contract *B*, where the traffic was exceptionally heavy, and where the surface was almost continually shaded, and therefore at all times somewhat moist, no difficulty whatever was experienced, and the blocks remained in an ideal condition. On the third project, called Contract *C*, which was exposed directly to the sun, but which had considerable traffic, some difficulty was encountered. A number of the blocks floated with pitch filler, and the surface of the pavement for a short distance at the beginning of the work very seriously loosened and deteriorated. After this difficulty was discovered, the blocks on the remainder of this contract were paved very closely, and driven as tightly as possible by means of heavy sledges. As an additional preventive a somewhat harder pitch filler was used, and on a portion of the contract the bottom of the joints were swept full of fine dust for a depth of ¼ in. in order to prevent pitch in the joints from running underneath the block. By this means it was felt that the probability of trouble from shrinkage of the blocks was very considerably reduced, but in order to permit of a slower adjustment of the pavement, the blocks on this project were kept covered with sand, which was sprinkled heavily throughout the summer. As a result of the combination of these precautions, the surface of the street on Contract *C* remained in good condition throughout the summer and winter of 1919, and was in excellent condition in the spring of 1920. As it later developed it was the sprinkling of this pavement which

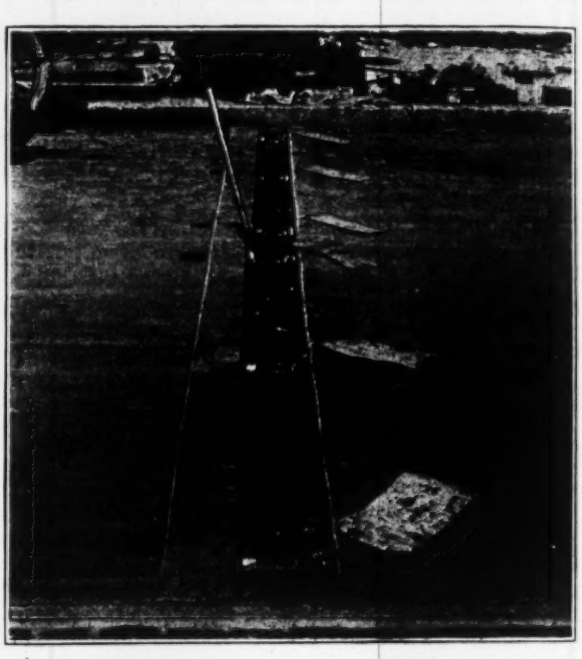


FIG. 1. ARRANGEMENT OF JACKSCREWS FOR TIGHTENING WOOD BLOCK PAVEMENT

was most effective in preventing shrinkage and floating during the summer of 1919.

In May, 1920, the sand covering had been washed from the street until only a slight resulting mat remained. During the first warm weather in May the pavement started bleeding, and although no bleeding had taken place in the first year, these blocks bled heavily during May and June. The street was sanded from time to time, but a series of heavy rains kept it washed fairly clean. In the dry weather of middle June the shrinkage of the blocks began, and went on in every respect as it had in Contract *A* the previous year. This resulted immediately in somewhat warped blocks, and in open joints, which permitted the blocks to rock under traffic, and which hastened the settlement of the pitch filler. As later investigation also showed, the bleeding of the blocks had very seriously cut the back filler, and there is no doubt that the filler in June, 1920, was much softer than the softest which had been put in the previous year. In fact, one sample of filler was found, the melting point of which was below 100 deg. This was an extreme case, and I should estimate that the melting

TABLE I. PROGRESSIVE MOVEMENTS OF POINTS 5-FT. APART IN DIRECTION OF TRAFFIC—FEET

Distances Out from Jackscrews—Feet	July	9	10	13	14	15	16	17	19	20
	Temp. Deg. Min.	80	69	82	78	77	79	80	67	
	Max.	83	81	93	84	84	88	92	83	
5		0.06	0.22	0.39	0.54	0.65	0.92	1.13	1.16	1.26
10		0.03	0.17	0.32	0.47	0.57	0.85	1.05	1.08	1.18
15		0.01	0.11	0.25	0.39	0.48	0.74	0.94	0.98	1.06
20		....	0.08	0.21	0.34	0.42	0.67	0.87	0.91	1.00
25		....	0.05	0.16	0.29	0.37	0.62	0.80	0.85	0.93
30		....	0.04	0.13	0.25	0.32	0.55	0.74	0.79	0.85
35		....	0.02	0.10	0.20	0.27	0.50	0.68	0.72	0.80
40		....	0.01	0.07	0.17	0.23	0.45	0.61	0.67	0.73
45		....	....	0.05	0.14	0.20	0.40	0.58	0.61	0.66
50		....	....	0.03	0.11	0.16	0.34	0.51	0.55	0.60
55		....	....	0.03	0.09	0.14	0.30	0.46	0.50	0.55
60		....	....	0.02	0.08	0.12	0.26	0.42	0.46	0.50
65		....	....	0.01	0.05	0.09	0.21	0.34	0.40	0.44
70		....	....	....	0.04	0.06	0.16	0.31	0.35	0.38
75		....	....	....	0.01	0.04	0.04	0.24	0.27	0.31
80		....	....	....	....	0.01	0.01	0.20	0.23	0.25
85		....	....	....	....	....	....	0.18	0.22	0.23
90		....	....	....	....	....	....	0.15	0.19	0.20
95		....	....	....	....	....	....	0.11	0.17	0.18
100		....	....	....	....	....	....	0.07	0.12	0.15

TABLE II. SHORTENING OF SPACES BETWEEN 5-FT MARKS—IN DIRECTION OF TRAFFIC—FEET

Distances Out from Jackscrews—Feet	July	9	10	13	14	15	16	17	19	20
	Temp. Deg. Min.	80	69	82	78	77	79	80	67	
	Max.	83	81	93	84	84	88	92	83	
5		0.03	0.05	0.07	0.07	0.08	0.07	0.08	0.08	0.08
10		0.02	0.06	0.07	0.08	0.09	0.11	0.11	0.10	0.12
15		0.01	0.03	0.04	0.05	0.06	0.07	0.07	0.07	0.06
20		....	0.03	0.05	0.05	0.05	0.05	0.07	0.06	0.07
25		....	0.01	0.03	0.04	0.05	0.07	0.06	0.06	0.08
30		....	0.02	0.03	0.05	0.05	0.05	0.05	0.05	0.05
35		....	0.01	0.03	0.03	0.04	0.05	0.07	0.05	0.07
40		....	0.01	0.02	0.03	0.03	0.05	0.03	0.05	0.07
45		....	....	0.02	0.03	0.04	0.05	0.07	0.06	0.05
50		....	....	0.00	0.02	0.02	0.04	0.05	0.05	0.05
55		....	....	0.01	0.01	0.02	0.04	0.04	0.04	0.05
60		....	....	0.01	0.03	0.03	0.05	0.08	0.06	0.06
65		....	....	0.01	0.01	0.03	0.05	0.03	0.05	0.06
70		....	....	....	0.03	0.02	0.02	0.03	0.08	0.07
75		....	....	....	0.01	0.03	0.03	0.04	0.04	0.05
80		....	....	....	....	0.01	0.01	0.02	0.01	0.02
85		....	....	....	....	....	....	0.03	0.03	0.03
90		....	....	....	....	....	....	0.04	0.02	0.02
95		....	....	....	....	....	....	0.04	0.05	0.02

IN DIRECTION AGAINST TRAFFIC—FEET

Distances Out from Jackscrews—Feet	July	9	10	13	14	15	16	17	19	20
	Temp. Deg. Min.	80	69	82	78	77	79	80	67	
	Max.	83	81	93	84	84	88	92	83	
5		0.05	0.25	0.44	0.60	0.76	1.02	1.14	1.19	1.30
10		0.03	0.20	0.37	0.53	0.68	0.93	1.05	1.12	1.28
15		....	0.15	0.31	0.46	0.61	0.85	0.94	1.02	1.13
20		....	0.12	0.26	0.40	0.54	0.77	0.87	0.94	1.03
25		....	0.09	0.20	0.33	0.46	0.68	0.78	0.85	0.94
30		....	0.07	0.16	0.29	0.41	0.61	0.71	0.80	0.86
35		....	0.05	0.12	0.24	0.35	0.55	0.64	0.73	0.79
40		....	0.04	0.10	0.21	0.32	0.50	0.59	0.68	0.73
45		....	0.03	0.08	0.18	0.27	0.44	0.52	0.60	0.66
50		....	0.01	0.05	0.14	0.22	0.39	0.45	0.54	0.59
55		....	....	0.05	0.12	0.19	0.34	0.40	0.49	0.53
60		....	....	0.04	0.10	0.17	0.30	0.36	0.45	0.49
65		....	....	0.03	0.08	0.13	0.25	0.30	0.39	0.42
70		....	....	0.02	0.06	0.11	0.21	0.26	0.34	0.36
75		....	....	0.01	0.04	0.08	0.16	0.21	0.29	0.31
80		....	....	....	0.04	0.07	0.14	0.18	0.25	0.27
85		....	....	....	0.02	0.04	0.09	0.12	0.20	0.21
90		....	....	....	0.01	0.02	0.05	0.07	0.15	0.16
95		....	....	....	....	0.01	0.02	0.05	0.11	0.12
100		....	....	....	....	....	0.01	0.02	0.09	0.09
105		....	....	....	....	....	....	....	0.06	0.06
110		....	....	....	....	....	....	....	0.05	0.05
115		....	....	....	....	....	....	....	0.03	0.03
120		....	....	....	....	....	....	....	0.02	0.02

IN DIRECTION AGAINST TRAFFIC—FEET

Distances Out from Jackscrews—Feet	July	9	10	13	14	15	16	17	19	20
	Temp. Deg. Min.	80	69	82	78	77	79	80	67	
	Max.	83	81	93	84	84	88	92	83	
5		0.02	0.05	0.07	0.07	0.08	0.09	0.09	0.07	0.02
10		0.03	0.05	0.06	0.07	0.07	0.08	0.11	0.10	0.15
15		....	0.03	0.05	0.06	0.07	0.08	0.07	0.08	0.10
20		....	0.03	0.04	0.07	0.08	0.09	0.09	0.09	0.09
25		....	0.02	0.04	0.04	0.05	0.07	0.07	0.05	0.08
30		....	0.02	0.04	0.05	0.06	0.06	0.07	0.07	0.02
35		....	0.01	0.02	0.03	0.03	0.05	0.05	0.05	0.06
40		....	0.01	0.02	0.03	0.05	0.06	0.07	0.08	0.07
45		....	0.02	0.03	0.04	0.05	0.06	0.07	0.06	0.07
50		....	0.01	0.00	0.02	0.03	0.04	0.05	0.05	0.06
55		....	....	0.01	0.02	0.02	0.04	0.04	0.04	0.04
60		....	....	0.01	0.02	0.04	0.05	0.06	0.06	0.07
65		....	....	0.01	0.02	0.02	0.04	0.04	0.05	0.06
70		....	....	0.01	0.02	0.03	0.05	0.05	0.05	0.05
75		....	....	0.01	0.00	0.01	0.02	0.03	0.04	0.04
80		....	....	....	0.02	0.03	0.05	0.06	0.05	0.06
85		....	....	....	0.01	0.02	0.04	0.05	0.05	0.05
90		....	....	....	0.01	0.01	0.03	0.02	0.04	0.04
95		....	....	....	....	0.01	0.01	0.03	0.02	0.03
100		....	....	....	....	....	0.01	0.02	0.03	0.03
105		....	....	....	....	....	....	....	0.01	0.01
110		....	....	....	....	....	....	....	0.02	0.02
115		....	....	....	....	....	....	....	0.01	0.01
120		....	....	....	....	....	....	....	0.02	0.02

point would not generally average below 130 deg., or a softening of 20 deg.

In the first experiment tried, the blocks were moved for a distance of 75 ft. on each side of the cut. Later, with favorable weather conditions, it was found possible to move the blocks for a distance of 125 ft. on each side. The attempt was not made to force the movement of the blocks, but merely to keep a uniform pressure on the jack-screws in an amount which would be easily secured by the use of a long-handled wrench, and which could be followed up from jack to jack as fast as the flow of the pitch under the blocks released the pressure on the screw. The rate of progress, therefore, varied almost directly with the temperature of the pitch bed under the blocks, and this was determined by the general temperature and the amount of sunshine. The direct effect of the hot sunshine in accomplishing these results was very noticeable. An hour of this condition was worth several hours of a hot, cloudy day in facilitating the movement of the blocks.

A typical record of this movement—Opening No. 5—is shown in Tables I and II. The figures represent the movement, at points laid out in advance of the work, at each 5-ft. interval from the center of the jacks. Thus, on the first day, the points 5 ft. each side of the jacks moved, respectively, 0.05 and 0.06 ft. On the second day the total movement had become 0.25 ft. and 0.22 ft., and so on to the completion of this work, when the total movement secured on one side was 1.3 ft. and on the other 1.26 ft., or over 30 in.

The movement was primarily a closing up of open joints, and very little compression was put into the timber at any time. This is noted from Table II, which indicates the closing up of the spaces, between 5-ft. points. It will be noted that this closure in the vicinity of the jacks amounted to from 0.06 to 0.09 ft. for each 5 ft. of pavement, while at the extreme of the movement this reduced itself to practically zero.

Undoubtedly, by carrying on the operations for a greater length of time closures of as much as 0.06 ft. could have been secured 100 ft. from the jacks, but at this stage of the operations the progress was slow, and it was found more economical to remove the apparatus and to replace it further down the street in such position as to move the blocks back up to the point reached in its first location. The fact that no permanent compression was put into the pavement was indicated in several instances where, on the removal of the jacks, no tendency was shown in the blocks to creep back into the open slot of the pavement. In other cases, however, a slight pressure had existed, and the hole was closed up as much as an inch while it was being repaved.

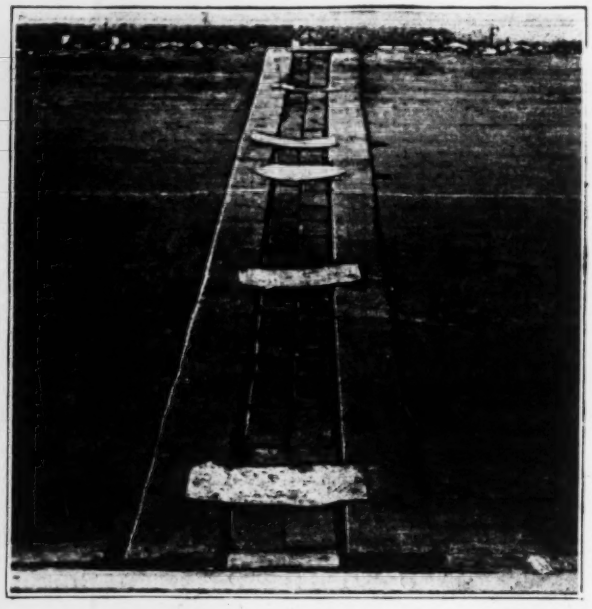


FIG. 2. JACKSCREW OPENING TEMPORARILY FILLED TO PASS TRAFFIC

Another very interesting phenomenon in connection with the operation of the jacks is shown by the relative motion of the opposite sides of the opening. As will be noted in Fig. 2, this work was all done under traffic. This was considered desirable, as it kept the blocks ironed out during the tightening process, and undoubtedly had a considerable effect in the smoothing out of the pavement. Traffic on this side of the street is from north to south, and at each opening the movement of the blocks towards the south, or with the traffic, was much less than the movement to the north, or against the traffic. In some cases, the movement with the traffic was only 50 per cent of the reverse motion. This was undoubtedly due to the tractive effort of the tires in pushing the pavement in the direction opposite to that taken by the vehicles. In Opening No. 5, however, the motion in the two directions was nearly equal, and this was accounted for by the fact that this opening was within 100 ft. of the intersection of a main thoroughfare, and that probably as many vehicles had occasion to stop within the area considered as there were vehicles which passed over without stopping, and that the braking effect of the stopping vehicles counteracted the tractive effort of those passing on.

In all, three sets of apparatus were used, and sixteen cuts were made in the pavement. In every instance the movement from adjacent jacks was made to come to a common point, so that when the whole of this pavement was tightened up in this manner the amounts of closure varied directly from the jacks to the intermediate points. The smallest amount of spread secured with any installation was 0.59 ft., and the greatest amount was 2.56 ft. The total of sixteen openings amounted to about 23½ ft., which was the length of the new wood block pavement inserted in the street. This is slightly less than 1 per cent of the original wearing surface, or about half of the measured shrinkage of the block.

The effect of this work on the pavement surface is noticeable in both Figs. 1 and 2, which are shown during the operation of tightening, as compared with the condition of Fig. 3. The surface has also improved somewhat since the removal of the jacking apparatus, and it appears that the effect of traffic is very beneficial to the surface after the blocks have been made to fit tightly together, just as the traffic was a factor in increasing the roughness of the pavement when in the condition shown in Fig. 3.

The pavement surface is now highly satisfactory, the result was accomplished without inconvenience to traffic, without the removal of any of the old pavement except that necessary to insert the jacks, and with practically no loss of blocks whatever. The entire work of tightening 7,500 sq.yd. was carried out with a force of one paver, a helper who adjusted the timbers, inserted the apparatus, repaved the wood blocks after the removal of it, and of one inspector who handled the wrench and kept pressure on the timber at all times. The work was interrupted for several days at a time by spells of cool weather, and the paver and helper were, during this time, used on miscellaneous adjustments and on the repair of high spots in the pavement. The work was started on June 14 and was completed on Sept. 14, at a total cost of about $1,200 for labor and supervision, and about $200 for new materials, a total of $1,400 or about $0.19 per square yard as compared to the $1.25 per square yard, the cost of relaying blocks.

As the street was originally paved, the blocks were

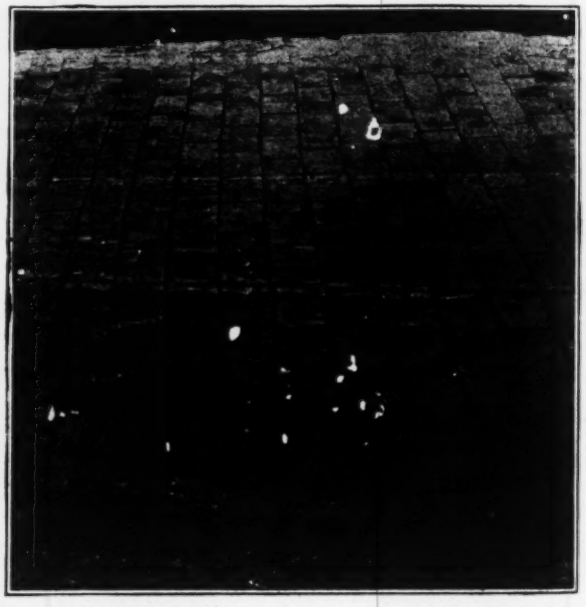


FIG. 3. CONDITION OF PAVEMENT BEFORE TIGHTENING

laid tightly together, but were at their maximum size, and the shrinkage of two to three per cent which took place in the pavement left them entirely loose. The blocks in the shrunken condition have again been placed tightly together, so that the joints are wood to wood, or separated by a small amount of original filler which was squeezed up during the tightening operation. The blocks will probably take up some moisture during the wet season, and some internal pressure will develop, but this should not be sufficient to cause a blow-up at any point, and, in fact, is exactly the condition which was customary in the laying of wood blocks under the old specification for sand cushion and sand filler.

The writer has come to the conclusion that the shrinkage of wood blocks in the St. Louis climate in the last two years has been due to a great extent to the green condition of the timber treated, resulting in a block of maximum size on delivery, and to an equally great extent, in the specification for laying these blocks on a pitch bed and with a bituminous filler. Under this specification, the block is surrounded, except on its upper surface, with bitumen, and cannot take up any appreciable amount of water, while it can evaporate freely from its upper surface. This certainly must result in the lowering of the moisture content, and in establishing a normal moisture content much lower than that noted at the time of treatment, and, incidentally, much lower than the normal moisture content of the wood block pavements laid under the old specifications.

With the combination of expanded block on delivery and a thoroughly sealed street, shrinkage appears to be inevitable. The method of laying must take this into account by putting initial compression into the pavement surface as we are doing in St. Louis with our new work this year, or we must be prepared to return to the work and do the necessary tightening after the shrinkage takes place.

Slag Aggregate Makes Satisfactory Concrete

Tests made in 1916 and 1917 by the United States Bureau of Standards on the value of slags for fine and coarse aggregate have recently been reported. The concrete so made seems to be of an equal strength with concrete made from ordinary gravel. In the tests concrete of satisfactory local sand and gravel was made at the same time of the same cement and mix as the concrete using crushed slag furnished by three commercial companies. The consistency in all the concretes was the same as judged by eye. The slag was crushed to coarse and medium sizes, the former being fairly uniformly graded from ¼ to 1½-in. openings, through the larger of which 100 per cent passed, the latter being all through a ½-in. screen, 57.4 per cent through a ¼-in., and 6.2 per cent through a 0.85-in. screen, these figures of course being for only one of the specimens and being merely typical. The weight of the gravel was approximately 100 lb. per cubic foot, while the slag was between 70 and 85. The various slags had a sulphide sulphur content varying between 0.04 and 1.43 per cent. The strength of the various specimens varied according to the different times and the different mixtures of which they were made, but quite uniformly the crushed slag aggregate made stronger concrete than the Potomac gravel aggregate. The conclusions of the bureau are as follows: Crushed slag as a coarse aggregate produced concrete of as high a strength as gravel. The tests were not extensive enough to determine the durability of the slag, but to the extent of the tests there were no signs of disintegration due to the sulphide sulphur or other causes. Slag sand, because of its lack of fine material, does not produce easily workable concrete when used as a fine aggregate. If it must be used its working qualities can probably be improved by the addition of small amounts of fine sea sand, hydrated lime or other fine material. Provisions in specifications for slag aggregate calling for a maximum sulphide sulphur content of 1.5 per cent and a minimum weight per cubic foot of 70 lb. are tentatively recommended.

Experience with Slab Pavements at Minatare Dam

Break Repaired Temporarily with Sacks of Gravel Placed Shingle Fashion—Sills Under Hinge Joints for Repairs

FOR protection of a 150-ft. break in a concrete slab pavement of the Minatare Dam on the North Platte Project of the U. S. Reclamation Service, bags of gravel were used as a revetment until the slabs could be replaced. From the experience gained from this break and former failures in 1916 the engineers in charge concluded that the material of the earth dam may be prevented from being sucked out through the joints and cracks by placing concrete ribs on sills under the joints between courses, the latter to extend up and down the slope in an unbroken sheet. Such precautions are deemed unnecessary where a 12-in. layer of gravel or spalls is placed under the slab for a foundation. This last break occurred in April and W. H. Fisher, assistant engineer, tells of it in the September *Reclamation Record*. The proposed reconstruction is described also by Andrew Weiss, project manager, who gives the experience of the Service at several other points. Extracts from both papers follow:

The reservoir water level at the time of the storm in April was 3 ft. below that of maximum capacity. The waves reached a maximum height between crest and trough of 5 to 7 ft. and the face of the dam received almost the full effect of the waves. The slope on the reservoir side of the dam is 2½ : 1 from the base of the dam to the elevation of maximum high water. Above this point the slope is 2 : 1 to the top of the earth dam for a vertical height of 15 ft. A concrete parapet wall surmounts the top of the concrete slab 1 ft. above the top of the dam, thus affording a total freeboard above high-water level of 16 ft. The effective freeboard at the time of the storm was therefore 19 ft. The concrete facing was made up of 8-in. rectangular slabs 10 ft. x 20 ft. with the 20-ft. dimension up and down the slope. These slabs were not reinforced except for a few short bars placed near the horizontal joints of the slabs to strengthen them in transmitting the component of slab weights lying along the slope. The effect of this reinforcing is practically negligible as far as the strength of the slabs is concerned. The horizontal joints between slabs are rounded, but the transverse or sloping joints are straight vertical joints. All joints have one thickness of ruberoid roofing inserted to separate adjacent slabs. The width of the break measured along the dam was about 38 ft. extending from a slab joint 9 ft. above the maximum high-water level of the reservoir to a line roughly 6 ft. below the same datum or 3 ft. below the water level at the time of the storm. The slabs above the high-water line were not badly broken, but for the most part were simply dropped down or left standing on a steep slope against the nearly vertical earth face left at the back of the washout along the dam. The position in which these slabs were left prevented a more extensive earth washout before the storm subsided. The lower portions of the slabs seem to have been badly cracked or broken by the violence of the storm and were shoved out over the unbroken pavement below, together with the earth filling washed out. A total of about 550 cu.yd. of the earth filling of the dam was removed during the storm, the major portion of which was dropped onto the submerged paving below the break.

During the fall of 1919, when the reservoir was empty, a series of tests was conducted to determine whether any hollow spaces existed beneath the slabs near the high-water line. Holes 4 in. in diameter were chiseled through

the slabs at what appeared as likely spots for investigation, but no hollows of any consequence were found, the largest being not over 2 in. deep and 3 or 4 in. wide.

The Minatare reservoir has been in service five years. During that time numerous cracks have developed in the concrete facing, and several of the transverse or sloping joints have opened up to some extent. Most of these cracks or openings are negligible but some have reached a critical width of ¼-in. so that with violent wave action it is possible and indeed probable that some of the underlying material, which is mostly coarse sand, has been washed out, particularly along those cracks running up and down the slope. There was evidence of this along one of the sloping slab joints, which was exposed by the break, where a small pocket about 3 in. deep was visible directly under this joint at a point at least 5 ft. above the water level. Such pockets would probably reach a maximum size at or near the water level, and it is conceivable that when such a pocket had reached a certain size the impact effect of heavy masses of water falling on such a slab might ultimately cause a settlement of one edge of the slab, especially should there happen to be a longitudinal crack in the slab that would readily allow such settlement to take place. Cracks might then open up to such a degree as to wash out still more material, until actual undermining would take place, and the slab would drop and expose the underlying material to direct wave action.

Temporary repairs of the break were started two days after the storm by a force of six laborers breaking up the large pieces of dislodged concrete slabs with gads and sledges. The pieces were left as large as practicable for handling and were then built into a bulkhead or wall along the lower edge of the break which was under 2 to 3 ft. of water. This wall had a minimum thickness of 3 ft., and was built as tight as possible, using only the rough broken pieces of slab. There was enough broken slab available to carry this wall to a height of about 3 ft. Coarse gravel was dumped on the crest of the dam immediately above the break. A single pulley block was anchored near the water line with a team on one end and a mormon scraper on the other. The team was kept on the dam and two men handled the scraper, pulling the gravel over the parapet wall and down into the cavity, which was thus filled to within 2 ft. of the original surface of the paving. A gang was then started filling sacks with gravel and placing them shingle fashion, starting at the bottom. First a layer was made of burlap sacks and then two layers of cement sacks on top. These sacks extended well over the top of the wall.

Proposed Reconstruction

A similar but less serious failure of a portion of the paving of Minatare dam occurred in August, 1916. Its cause was thought to be primarily the settlement of the dam foundation, which was noticed during the time of the building of the dam, and caused a series of horizontal cracks in the paving. In some cases these cracks became sufficiently enlarged to permit the wave wash to suck some of the fine material from under the blocks, thereby partly undermining them. As this process advanced the facing blocks finally lost support, and horizontal cracks opened until failure occurred.

Numerous tests were made over the entire facing, especially near the places where these horizontal cracks had developed, to determine the extent and location of hollows which had formed or were in process of forming. A compressed-air grouting machine was installed, the defective places in the face being tapped with holes and grout forced into place to refusal. The broken portion of the pavement was replaced after the close of the irrigation season in 1916 and no further deterioration was noted on the facing until the recent break.

The repair of the present break is thought to be a simple matter of replacing the broken paving with slabs of the original thickness 10 ft. in width and either of continuous length or with hinge joints horizontally placed, as originally built. This paving should be reinforced w[ith] ¼-in. square bars, slab reinforcement being placed horizo[n]tally as well as along the slope. Under the joints it [is] suggested that concrete sills 6 in. thick and 12 in. wide b[e] placed to prevent the further removal of fine material b[y] sluicing from underneath. This means of prevention wa[s] adopted in the case of the Pruitt Reservoir, illustrated o[n] p. 103, vol. 77, *Trans.* Am. Soc. C. E., 1917.

An effort will also be made to locate any further hollow places which may have formed under the paving elsewhere, particularly where any other cracks are noted through which the sand may have been drawn by the wave action.

The behavior of the Minatare dam facing suggests that wherever it is desirable to omit the screened gravel layer between the concrete facing and the body of the dam the facing should be reinforced and placed in continuous layers from bottom to top, or in slabs of not less than 20 ft. length with hinge joints, and that the joints between these courses be underlaid with reinforced-concrete stringers or ribs, which would prevent the sucking of sand from under these slabs. It is very seldom that unscreened gravel can be obtained containing a sufficient proportion of coarse material to resist effectively the wave wash. In consequence a crack of any size, even a temperature crack, becomes a source of danger unless the underlying material is in some manner protected from resulting suction.

There has been so far practically no trouble whatever on this project with dam facings constructed of layers of unscreened material overlaid with screened gravel or spalls, on top of which was placed loose rock paving. This method of paving has proved an excellent protection at the Pathfinder Dike, and also at Lake Alice, where no repairs have been necessary since those dikes were built seven years ago.

Two Sources of Brittleness in Steel Rails

According to a review of the present status of our knowledge of brittleness in steel rails, presented as a report to the Rail Committee of the American Railway Engineering Association by M. H. Wickhorst, engineer of tests to the Rail Committee of the association, brittleness is of three kinds, showing itself in fracture starting (1) from an area of segregated carbon and phosphorus, (2) from a minute internal crack, and (3) from a seam (narrow groove) in the bottom face of the base. Both of the first two causes are responsible for failures which originate in the head or develop when the rail is so bent as to put the head in tension. Fractures developing when the base is in tension are due to the first and third causes only. While brittleness due to segregated areas exist simultaneously in head and base of the rail, the segregation generally extends farther into the head than into the base, and therefore is more readily detected by bending with head in tension than with base in tension. Mr. Wickhorst consequently recommends that in acceptance bend tests to determine ductility the head should be put in tension. Bend tests show the same ductility whether the drop testing machine or a hydraulic bender be used, but in the former case the impact is likely to break off the flange in case there are seams in the base, which would defeat the object of showing the actual ductility of the metal. The hydraulic bender, which always gives normal tension breaks, is therefore recommended as preferable to the drop testing machine. Segregation of carbon and phosphorus causes not only longitudinal but also transverse brittleness, the latter producing split heads when the top part of the head flows under the rolling action of wheels. The same effect may be due to large amounts of non-metallic inclusions, commonly manganese sulphide.

Boston Molasses-Tank Trial: The Case for the Defense

(A bomb caused the notorious molasses-tank collapse, is maintained by the defense—the company owning the tank—in the trial of damage suits now in progress. All reports of the accident directly after it occurred, and the facts developed in the official inquest by the municipal court, indicated that the tank was seriously defective, and was so highly overstressed that its failure in service was to be expected. Because of the opposite character of the explanations of the collapse, a summary of the facts and expert opinions advanced by the defense has been secured by our technical correspondent—EDITOR.)

TRIAL of about 100 damage suits brought against the United States Industrial Alcohol Co., owner of the molasses tank in Boston which collapsed on Jan. 15, 1919 (see *Engineering News-Record*, Feb. 13 and May 15, 1919, pp. 353 and 974), killing 12 persons, has now been progressing before Col. Hugh W. Ogden as master for nearly seven weeks. The plaintiffs contented themselves in direct evidence with proving the ownership of the tank and the damages done. Such expert testimony relating to the structural soundness of the tank or other matters as they may have to present is apparently being held back to be submitted in rebuttal of the defendant's case. The interesting features of the case thus far made public are, therefore, included in the defendant's case, which now seems to be nearly complete.

The defense is that the failure of the tank resulted from the explosion of a bomb or its equivalent maliciously placed in the tank and thus from causes over which the company could have no control. Testimony has been presented to show that the company had had labor troubles involving damage to property; that two of its nearly new tank steamers have been mysteriously lost at sea; and that posters threatening damage to property were found in Boston not far from the tank within a comparatively short time before the failure. They also produced a witness living in a house near the site of the tank who testified that she happened to be on the roof of her house at the time of the failure and that, a few seconds before the tank gave way, she saw smoke rising from the ventilator in the roof of the tank. Some of the experts testified that they could find no cause for this smoke other than a burning fuse, presumably attached to an explosive.

Frank E. Sherry, a civil engineer of Boston, testified to measurements and plans of the locality of the tank and of the broken pieces, and also to his examination of the locality shortly after the failure. All of the parts were carefully measured and their original position in the tank determined so that it could be seen how far and in what directions the various parts were carried by the disruptive force. The most significant part of his testimony relates to a section of the bottom and of the lower course of plates of the tank, which was torn in an irregular and jagged manner through a point in the bottom about 11 ft. from the side, approximately vertically through the side plate, the tear going through the manhole and then below the riveted joint near the top of the lower course. This section of the tank, which weighed about 2½ tons, was found at a distance of 180 ft. from its original location.

The tank was 50 ft. high and 90 ft. in diameter, surmounted with a conical steel roof on light steel trusses. The roof plates were about ¼ in. thick and there were four trap doors or manholes in the roof. These were not fastened and could be easily opened. A 6-in. pipe ventilator was set in the apex of the roof. There was a spiral stairway leading from the ground to the roof and, although a locked gate was provided at the foot of the stairs, it was not difficult to scale this, and it is stated that workmen frequently did climb to the roof of the tank for the purpose of watching interesting occurrences in the harbor.

A portion of the bottom nearest the center of the tank showed a decided depression, as did other sections of the bottom plates at this locality. Further, there was a marked hollow or depression in the cushion upon which the tank had been set immediately under this point in the bottom. This cushion originally consisted of a mixture of dry cement and sand. It had partially set, so that over most of the base it remained in place when the molasses flowed out, although it was brittle and comparatively easily broken. At this particular point, however, a piece perhaps 3 ft. across had been entirely washed away by the flowing molasses. The way the bottom plates were bent and this depression in the cushion indicated to Mr. Sherry that an explosion took place in the tank at this point of the bottom.

Prof. George E. Russell, of the Massachusetts Institute of Technology, Prof. George F. Swain, of Harvard University, and Lewis E. Moore, engineer of the Massachusetts Public Utility Commission, testified as hydraulic and structural experts, and all to substantially the same effect: that the tank was structurally safe, although the factor of safety was materially less than they would have provided. The tank did not fail at its weakest point, which was in a lap joint in the second course of plates, where the stress was computed at about 31,800 lb. per sq.in., allowing an efficiency of 66 per cent in the riveted joint. The greatest stress at the point of rupture through the manhole was computed at about 28,000 lb. per sq.in. The tensile strength of the steel was found by test of a considerable number of pieces to be 55,000 to 56,000 lb. per sq.in. The actual weight of the molasses used in computing the stresses was 88 lb. per cu.ft.

The section of the bottom and side referred to above as carried to a distance of 180 ft. from its original position could have been propelled to this point by flowing molasses only in case a velocity of 52 ft. per second were developed, according to Professor Swain's testimony, whereas he found that under the actually existing pressure of 29.8 lb. per sq.in. the greatest velocity could not have exceeded 40 ft. per second.

The factor of safety was computed as approximately 1.6. Although the engineers agreed that this was lower than the best engineering practice would dictate, they saw no reason to believe that structural weakness was the cause of the failure. The company owns a number of similar tanks which are located at various points along the Atlantic seaboard. All of them have been examined and several have been found to be under greater stress than the one which failed. All of them, however, have been in use for considerable periods of time and show no signs of failure.

The fractures did not show any signs of gradual failure, but in every case the breaks were sharp and no drawing down of the metal could be discovered. There

were also no signs of fatigue such as might have resulted from repeated filling and emptying. The tank had been filled 13 times previous to the final filling, which was absolutely inadequate to cause fatigue in the metal and yet was more than enough to develop structural weakness if any existed. Mr. Moore testified that hundreds of thousands of repetitions of stresses beyond the elastic limit were necessary to cause failure due to fatigue.

The plates of the lower course in the vicinity of the point where the explosion is presumed to have occurred showed a bulging about 1 ft. above the bottom varying from 1 to 4 or 5 in., and also five vertical cracks were discovered. In preparing for the case, the experts had two small tanks 9 ft. high constructed upon the company's property in Baltimore, and destroyed them by small charges of dynamite. In one case a charge of 8 or 9 ounces of dynamite was exploded at a distance of 3½ ft. from the wall of the tank, this explosion resulting in a decided wave of molasses, which went over the top of the tank, and bulging the upper plates. In the second case, 5 ounces of dynamite were exploded 1½ ft. from the side of the tank and the effect was almost exactly similar to that produced upon the Boston tank which failed.

A. L. Colby testified to the tests of the metal cut from the plates of the tank which were made at Lehigh University.

Mr. Wedger, explosive expert for the state of Massachusetts, gave testimony relating to various kinds of explosives and their effects, but during his testimony no one not connected with the case was allowed to be present, so that it is impossible to give at this time any account of his testimony.

Prof. A. H. Gill, of the chemical department of the Massachusetts Institute of Technology, testified to his examination of the tank on the day of the accident and stated that he did not at that time have any idea that the failure might have resulted from a high explosive inside the tank. He made his examination originally with the idea of finding whether failure could have resulted from fermentation, from the explosion of gas which might have formed or accumulated in the tank, or from an explosive placed outside the tank, and could find no evidence which would support any of these theories.

Dr. Gill testified to experiments carried on at Baltimore in a tank filled with molasses to a depth of 19 ft. to see whether smoke from a burning fuse would rise through molasses and be visible at the top. Eight pieces of waterproof fuse of two different brands were used. From the moment that they were lowered below the surface of the molasses, smoke began to bubble up to the surface, and it passed out from five 6-in. circular holes in the top of the tank. He believed that a single piece of such fuse burning in the tank which was destroyed would produce a smoke which could be seen rising from the ventilator of the tank.

The company intends to present evidence from Prof. Albert Sauveur of Harvard as metallurgist, and it is inferred from the references made in the testimony already given that this testimony will relate largely to the presence of the so-called Neumann bands in the fractured metal, and will state that such bands have never been found except in metal fractured by explosion or similar disruptive force. It is possible that one or two other experts may be put on, but it is not expected that any other new lines of evidence will be presented.

It is presumed that the plaintiffs will present evidence from their engineering experts in rebuttal, but no indication of the causes which they expect to assign for the failure has yet been presented.

Hospital Service and Expenses in Construction

From an address by Dr. J. P. Cleary on Sept. 29, at Milwaukee, Wis., before the Construction Section of the National Safety Council.

IN THE construction of a plant for the Cadillac Motor Car Co. in Detroit, Mich., the du Pont Engineering Co., from August, 1919, to August, 1920, employed about 17,000 men, the maximum at any one time being about 3,600, and the minimum about 800. During the year there were 4,490 injuries. Of that number 80 were major injuries, necessitating the loss of time. Of the 80 major injuries, four resulted fatally, one in permanent, total disability, and the remaining 75 caused a loss of approximately 14,752 working hours, an average of 196⅔ hours per major accident. In addition to the 4,490 first dressings or treatments administered, approximately 3,229 re-dressings were required, amounting to 7,719 treatments for injuries administered during the year.

It is estimated that 3,500 medical cases were treated, making a total of 11,219 treatments given during the year. During the influenza epidemic last winter an average of 30 medical cases were treated every day. Of the 11,219 treatments, perhaps 1,432 were required in cases of major injuries, leaving 9,787 treatments administered for minor injuries or illness.

The cost of equipping and maintaining the plant hospital for the period mentioned amounted to $6,800. Of this amount, $5,900 was applied to the treatment of minor inquiries and medical cases. Dividing this amount by the number of treatments, 9,787, shows an average cost of 60c. each. During this period of time, 15,765 men were examined by the medical department, and 15,615 were accepted and placed on the company's roll. In addition to this number there were approximately 2,500 men on the rolls of the subcontractors during this time, making a total of 18,115 men who were protected by the medical department during the year in question. This amounts to an expenditure of approximately 37c. for each man so protected. There were 366 penetrating wounds of the feet resulting from stepping on nails. Of that number two, or 0.54 per cent resulted in lost time.

Starting Engines on Locomotives

Locomotives with "booster" or starting engines on the trailing axles to give increased power in starting are being tried on the New York Central R.R. as a means of increasing the efficiency in handling heavy passenger and freight trains. A two-cylinder inclosed horizontal engine mounted at the rear of the frame of the trailing truck drives a pinion which through an idler drives a gear on the trailing axle. The engineman can put this "booster" in operation only when the reversing lever is in full position and the throttle is open. When he notches up the lever the booster is cut out automatically and is disengaged so that it cannot become a load on the locomotive when running at ordinary speeds. The additional weight is about 3,500 lb. but is said to be equivalent to increasing the adhesion weight of the engine by about 25 tons. It is intended to give an increase of 25 to 30 per cent in drawbar pull for different types of engines, thus insuring a steady and even start, which reduces damage to the engine and the cars.

The Transportation Engineer and Railway Economics

New Position Suggested to Insure Broader Views of Operating Problems and Their Economic Relations

CLOSE relationship of the engineer to railway transportation, or the operating department of railway service, is of sufficient importance to warrant wider official recognition and his appointment to a position of responsibility with regard to operation in which he will cover a broader field than that included in his more specific relation to construction and maintenance. This opinion was presented at a recent meeting of the Western Society of Engineers by J. A. Peabody, signal engineer of the Chicago & Northwestern Ry., who outlined some of the problems to be dealt with in the endeavor to increase the efficiency and economy of main line and terminal railway operation. Extracts from Mr Peabody's paper follow:

On every large railway there is a place for an engineer who, with proper assistance, should make a specialty of studying problems of transportation. In what part of the organization the engineer of transportation would best fit would vary on different railways, depending somewhat on the size of the railway, more on the amount and importance of the work undertaken, and most on the man. In every large railroad organization probably there is a man who, due to his experience, breadth of vision, good judgment and understanding of operating questions is eminently fitted for the work of transportation engineering.

To get the proper results he must be given freedom in developing his plans, and authority to call on any department for information. His position must be such that when his plans are completed he would naturally be called in consultation with such officials as the vice-president in charge of operation, who would give the reasons for the necessity for additional operating facilities, and the chief engineer, who would advise as to the best methods for the carrying out of the work. Where such a separate organization is formed there should be no fear that the transportation engineer would take prerogatives from any of the present department heads, engineering or otherwise.

Transportation engineering has a wide scope, because with the movement of both freight and passenger traffic to be studied there must come studies of the growth of communities, mining, farming, urban, suburban and industrial. The studies relative to handling of freight would include not only those relative to moving trains over the road, but also those of handling freight from the time it approaches the railway property to the time it is delivered into the cars or freight houses, and then the handling of cars into trains. The handling of passengers, baggage, express and mail should be analyzed in the same way, each of these being a problem and having a relationship to the others which is too often overlooked.

When the question of facilitating the movement of freight trains over the road is under consideration, how often is a definite analysis made to determine where interference occurs? Wellington mentions the necessity for proper location of passing tracks, but it was not until recent years that F. L. Dodgson developed a method of analysis to determine the proper location of passing tracks on a single track railway as described in the "Proceedings" of the American Railway Engineering Association for 1916, 1917 and 1919. An analysis and a complete plan for passing tracks of each division should be made, so that as money becomes available it may be so expended as to develop this complete plan.

With reference to the cost of operating trains there are several items which should be investigated. For instance, the cost of stopping a train and its relation to the cost of preventing that stop. There is no complete information on this subject and a brief paper which I wrote in 1905 is referred to even today, although the figures are entirely obsolete. The cost of operating trains, not for the railway as a whole but for each engine division, should be investigated in order to determine, if possible, how it may be reduced. The cost of switching in each yard and of handling engines at each terminal should be known, but in a recent investigation I have found that men who have given me the cost of switching in certain yards doubted their own figures.

Analysis should be made of conditions at each piece of congested main track to determine methods for increasing its capacity, together with the cost. The relative cost of operation under present and improved conditions also should be taken into consideration. This study would cover the location of passing tracks, the signaling and interlocking, the despatching system, reduction of grades and curvature and all other items that affect economical operation. Analysis of large freight terminals should be made to determine whether the handling of cars cannot be reduced materially. In a large terminal such as Chicago the consolidation of some yards would reduce the transfer work and, therefore, the several handlings of a car. It would also shorten materially the time required to deliver a car from the road to an industry. The methods generally used in the handling of l.c.l. and trap car freight in large terminals seem to be crude and a close study of this problem should produce methods resulting in quicker and cheaper handling.

In passenger service there are opportunities for materially decreasing the cost of operation, particularly in handling suburban traffic. The steam railway engineer is behind the electric railway engineer in his knowledge of what can be done in this direction. As to main line electrification there seems to be no doubt that many lines which now suffer from traffic congestion on account of their grades could increase their capacity greatly by electrification and probably decrease the cost of operation at the same time. My conclusion is that transportation problems will be solved only by the application of engineering knowledge, and that consequently the engineer has a better opportunity of showing his worth today than ever before.

Concrete Piers Hooped in Upper Part

In extending the King Edward Hotel, Toronto, Canada, foundations were constructed by excavating open wells to rock, about 35 ft. below street surface (or about 20 ft. below basement floor over most of the area), and filling these wells with concrete to form piers on which the steel columns of the building foot at basement-floor level. For the exterior columns, however, it was found more economical to carry up the concrete piers to a little above street-floor level and start the steel columns here. Because of the height of these exterior piers it was considered desirable to take special pains for insuring that the column load would be transmitted to the whole cross-section, it is stated in the course of a description of the work given by R. E. W. Hagarty in the *Canadian Engineer* of Aug. 26, 1920, p. 273. Below the steel I-beam grillage capping the pier, therefore, the concrete is reinforced with steel hooping (1 per cent of the cross-sectional area) for 6 ft. of height of the pier. At the 6-ft. level, it was considered, the stress would be thoroughly distributed over the pier cross-section; below this point no reinforcing is used. Mr. Hagarty, together with A. H. Harkness, carried out the engineering design of both foundations and superstructure of the building. The architects are Esenwein & Johnson, and Watt & Blackwell.

Building the American Niagara Power Extension

Details of the Dredging, Tunneling and Erection Operations on the War-Time 100,000-Hp. Addition to the Hydro-Electric Plant of the Niagara Falls Power Co.

Two earlier articles, Engineering News-Record, Sept. 23 and 30, 1920, described the structural and hydraulic features of the new extension to Station No. 3 of the Niagara Falls Power Co., which was started during the war, largely at the instance of the Government, to provide 100,000 hp. additional power at Niagara Falls. This article, by members of the construction staff of the company, completes the series describing this largest hydro-electric development now operating.

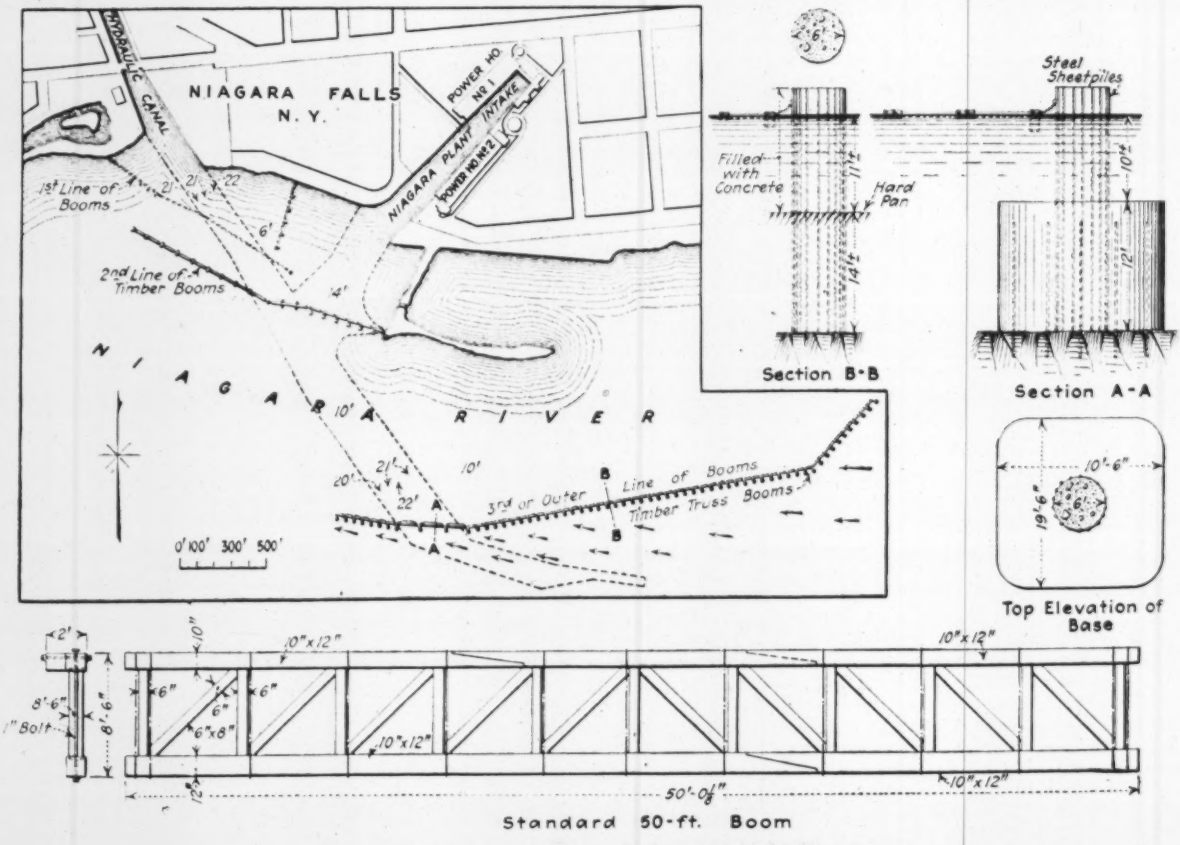
Enlarging Channels and Erecting the Station

By O. D. Dales
Construction Engineer, Niagara Falls Power Co., Niagara Falls, N. Y.

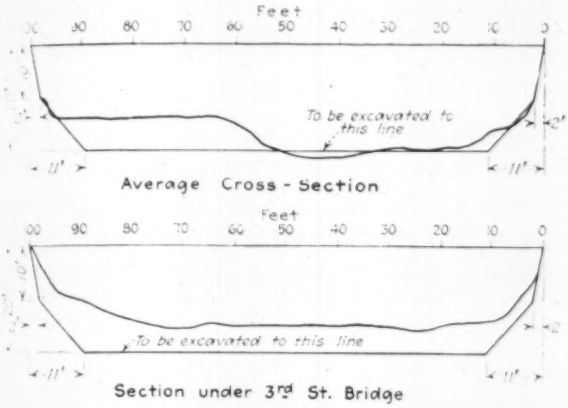
THE excavation of the 20-ft. channel from the head of the canal out into the Niagara River was through limestone for the first 1,000 ft. out from shore, the balance being in hardpan. The drilling was done by four drill boats with steam drills, the holes being placed on 4-ft. centers each way. Drilling was carried down to 3 ft. below the finished grade. The dredging was done by three dipper dredges with 6½-yd. buckets. The excavated material was placed in dump scows and towed out into the river just above the upper rapids, this class of excavation being similar to most dredging operations in large rivers. Disposing of the excavated material was rather difficult on account of the location of the dumping ground. Extreme caution was exercised by the contractor, but in spite of this one dump scow broke its two tow lines and floated down the Canadian channel to below the upper cascade rapids where it went aground on some rocks about 800 ft. from the Canadian shore. The two men on board were removed by the Fort Niagara life-saving crew by means of shooting a line out to the scow and bringing them ashore in the breeches buoy.

Piers for Booms

Most of the piers for supporting the two inner lines of booms were built in previous years and consist of concrete piers upon rock-filled timber cribs. These offered considerable resistance to the water so another type of pier has been developed which consists of Lackawanna steel sheet piling driven through the hardpan to the rock, which is from 6 to 15 ft. below the bottom of the river. These piers were built as 6 ft. diameter cylinders filled with concrete from the hardpan to the top. Details are shown on an accompanying drawing. This type of pier was used in the outer line of booms and were placed 50 ft. apart except in the deep dredged channel where they were 150 ft. apart.



ICE PROTECTION AT CANAL INTAKE ON NIAGARA RIVER



TYPICAL CANAL CROSS-SECTIONS

As there was no hardpan in the dredged channel the bottom of the 6-ft. cylinders were encased in 19-ft. 6-in. square concrete bases 12 ft. high. The past winter was very severe and a large amount of ice passed through the Niagara River thoroughly testing these piers and booms.

It has been the intention for a number of years to deepen the canal to 20 ft. below mean water level with a cross-section approaching as nearly as possible the typical section shown on the drawing herewith. Excavation had been carried on in the canal for a great number of years and up to 1912 had been excavated on the right-hand half, looking downstream, down to 20 ft. for nearly the entire length of the canal. The left-hand side of the canal was left at 14 ft. deep, while under the bridges it was approximately 14 ft. deep on both sides.

In order to have sufficient water available for the operation of the new plant along with the old plants it was necessary to excavate and clean up the canal to a depth of 20 ft. for the entire width approaching the typical section shown on the drawing. The excavation in the canal was a much more difficult task than that of the river. It was all in hard tough limestone, and the mean velocity of the water was 6 ft. per sec. which gave a surface velocity of 10 ft. per sec. when dredges and scows were in place. Eight bridges span the canal which vary from 6 to 15 ft. above the water surface. The drilling and blasting in such a severe current presented many difficulties, but was accomplished from a drill boat which was developed for this work.

Canal Drill Boats

The canal drill boat is equipped with five tripod drills operated by a crew of 12 to 15 men. Each drill has one runner and helper. The drill-boat bow is divided into 12 spaces 4 ft. apart and the boat moves back on ranges 4 ft. apart, making the spacing of the holes 4 x 4 ft. and they are drilled 3 ft. below finished grade.

The drill steel is 1¼ in. diameter and from 20 to 26 ft. long, depending on the depth of the water, with 1½- to 2½-in. bits. The steel drill rods are protected from the current by a main guide barrel of different lengths so as to keep it at least 3 to 4 ft. above the rock. The main guide barrel is 6 in. in diameter and has a 3-in. slit opening from the top to the bottom so that the steel drill rods may be pulled into place. At the bottom of this main guide barrel is a 1½-in. ring band to which a cable with a chain fastening is attached to the upstream end of the boat so as to keep the main guide barrel in a vertical position against the strong current. In order to protect the drill rods down to the top of the rock, a 4-in. diameter pipe and a 2¾-in. diameter pipe, one 6 ft. long and the other, or bottom piece, 5 ft. long, are made to telescope one another. This telescopic pipe is inserted into the main guide barrel and dropped to the bottom by a chain. There is a ½-in. round rivet stopper about 3 in. long which projects out from the side of the telescopic barrel about 6 in. down from the top and as the stopper slides down the slit in the main guide barrel it is stopped by the 1½-in. round cable band at the bottom of the main guide barrel which holds the telescopic barrel in place and which is lengthened or shortened by the telescopic action and weight of the pipe.

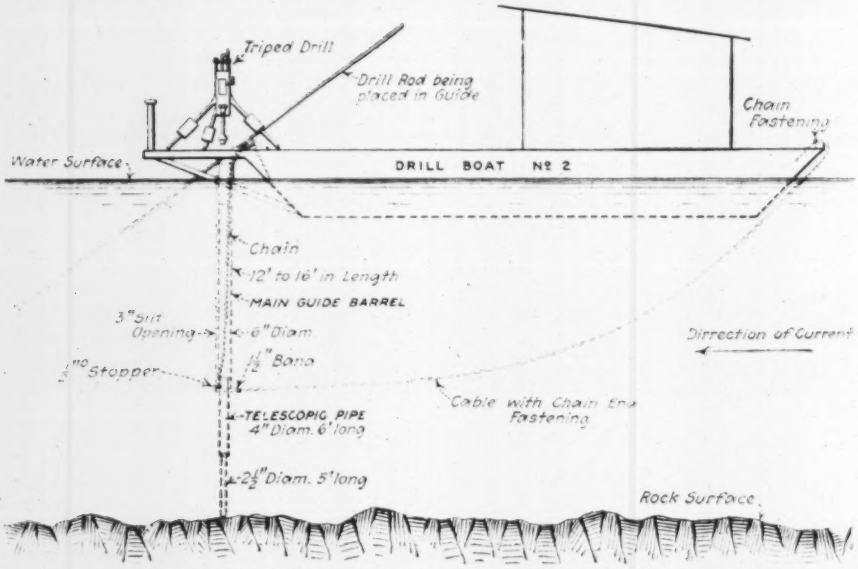
The drill rods are put into place by hand. They are first run about half-way out from the end of the boat and pulled back into the main guide barrel through the slit opening by two men and dropped to the top of the rock. As each two feet are drilled it is replaced by a longer drill rod.

ICE BOOMS IN ACTION ON NIAGARA RIVER MARCH, 1920

In blasting a 2-in. tin tube 20 ft. long is run down to the top of the hole through the guide pipe which keeps it from bending or breaking. The sticks of dynamite are put down the tube by a sectional lowering stick, the exploder stick being in the center of the charge which has the two wires attached and extend to the boat up the tube. Before the charge is fired the telescopic barrel is pulled up into the main guide barrel by the chain and let stand, also the tin tube is removed. The charge is set off by a battery and one hole is fired at a time. The kind of explosives used is 60 per cent Dupont and averages about 1 to 1½ lb. per cu.yd.

The ordinary 1½-yd. dipper dredge was used for dredging between the bridges. The dredging under the bridges was accomplished by two dipper dredges with extremely low cranes, one having a ¾-yd. and the other a 2-yd. dipper. These dredges were developed particularly for this work on the canal and have given very satisfactory results. One is shown in operation in one of the views.

The rock excavated from the canal was placed in steel skip boxes on board scows and floated down the canal under the control of a strong tug to the crusher plant at the canal basin where the rock was unloaded by derricks and crushed. From this point the rock was shipped by train and truck.



DRILL BOAT USED IN CANAL EXCAVATION

The drill boat and dredges built for this canal work were developed by R. R. Coddington, General Superintendent, from his many years' experience on this class of excavation. The contractor for the excavation in the river and canal is the Great Lakes Dredge & Dock Co.

Basin Excavation

The old canal basin which connected the canal with the inlet to Station No. 3 was 70 ft. wide and approximately 18 ft. deep. As the water for Station No. 3 Extension is taken from this basin, it was necessary to increase the carrying capacity of this basin which was done by increasing the width by excavating on the east side to a width of 117 ft. at the upstream end of the forebay inlet and gradually narrowing down to the old width of 70 ft. near the inlet to old Station No. 3. A curve of 143 ft. radius tangent to the north line of the canal and to this new basin line has made a very good entrance to the basin. With this enlargement of the basin the mean velocity of the water will be 4.5 ft. per second.

The top of the limestone rock lies about 8 ft. below the earth surface and about 2 ft. above the water surface in the basin. The earth was excavated with a ¾-yd. steam shovel. The rock was drilled and blasted in the dry down to about 2 ft. below the water surface by leaving a narrow ridge of solid rock as a cofferdam. The blasted rock was removed into dump wagons by the ¾-yd. steam shovel.

The next 11 to 13 ft. down was drilled and blasted in the dry and removed with a dipper dredge. The drilling was done with tripod drills, holes being placed about 4 ft. apart each way and along the side it was close drilled. These holes were 6 to 8 in. apart. It has been found that this close drilling in this class of rock gives a comparatively smooth wall for the side of the cut. In the excavation above water 0.8 lb. of 60 per cent dynamite per yard was used.

The lower 9 ft. of excavation for the basin enlargement was drilled and blasted from the drill boat and removed by the dipper dredge along with the deepening of the rest of the basin and in the same manner as the canal excavation.

Much of this basin enlargement was done in the summer and fall of 1918 when it was imperative on account of the war to keep all the old power plants running to their utmost capacity. The blasting up of the bottom of the basin, the ridge formed in front of the dipper dredge and the dredge with scow restricted the cross-sectional area so as to increase the velocity of the water very much. These conditions are shown in the view on p. 697, which was taken Oct. 19, 1918. The work was done in strips the length of the basin and sufficiently wide for the convenient operation of the dredge. Each strip was completed through before beginning another so as to reduce the cross-section as little as possible.

The forebay was built in rather cramped quarters, being separated from the basin by the basin road and between two large flour mills on the north and south, while on the west were busy switchtracks.

The old west wall of the canal basin, which is shown on p. 697, is built of concrete upon limestone. The top of the rock is at about water level. Under the roadway the inlets to the forebay have been built.

In the construction of the forebay this west wall with a wider strip of rock under it was at first used as a natural cofferdam for the excavation. As soon as the basin had been widened and deepened enough by dredging to allow a cofferdam to be built in it and also permit about 7,000 sec.-ft. of water to pass to old Station No. 3, a cofferdam was built. It was quite important that this cofferdam should restrict the flow as little as possible. With this in mind a narrow cofferdam was built of Lackawanna 12¾ x ⅜-in. straight web steel sheet piling in water varying in depth from 16 to 24 ft. The cofferdam consisted of 18 oval-shaped pockets. The size of a typical pocket in plan is 14 ft. 4 in. long, 9 ft. 10 in. wide at the mid-section, and 6 ft. 5 in. at the end where it joins the adjacent pocket. The outside row is composed of piles 24 ft. long and the inside row of 20-ft. piles.

In the construction, timber trusses were first built in lengths 28 ft. long and 8 ft. deep, using 12 x 12-in. timbers for the chord members and 8 x 8's for the diagonal members. These were built on the bank along side of the basin, launched, and floated into their place which was along the old concrete basin wall. Upon the cofferdam side of the truss were fastened timbers shaped to fit the outside of the pockets. After the truss was in place templets made of two thicknesses of 2-in. plank and the shape of the inside of the pocket were floated into place and the sheet piling placed around these templets and against the truss. Work was begun at the upstream end so as to take advantage of the current. As soon as the piling for one pocket was placed they were driven with a light

piledriver so as to drive them through any loose rock which might be on the bottom of the basin and cause them to fit as closely as possible to the solid rock. After this a diver went down into the pocket and closed up any openings that were found between the piling and the rock, using bags filled with concrete. Good rich concrete was then poured in the bottom of the pocket for a depth of about 2 to 3 ft. so as to properly seal the bottom and thoroughly fasten the cofferdam to the rock. The pocket was then filled up to the top with ordinary gravel, the pressure of this gravel causing the pockets to bulge slightly and completely tightening the joints. This made a very narrow cofferdam but it was thoroughly fastened to the rock at the bottom by concrete and at the top supported by two stiff wooden trusses.

In excavating the rock back of this old basin wall as much solid rock was left as possible between the piers to act as pilasters for supporting the old concrete wall which was receiving the load thrust upon it through the trusses from the top of the cofferdam.

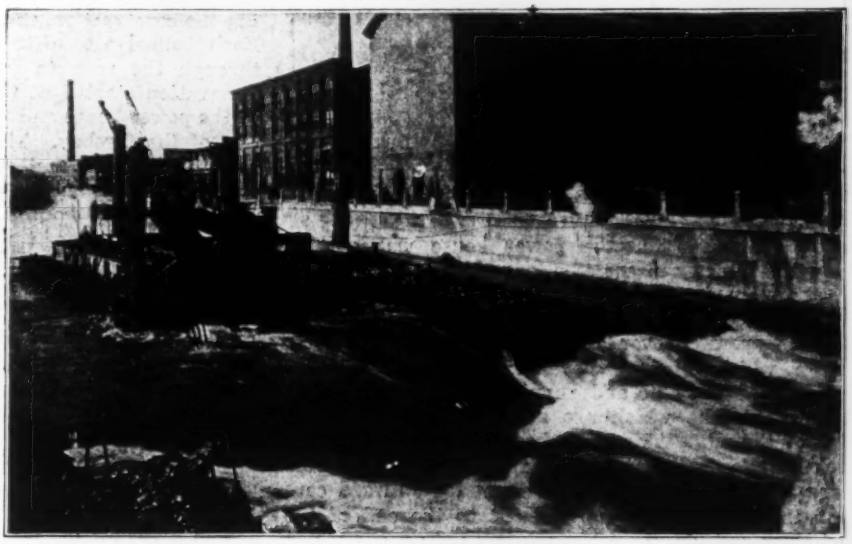
As soon as the piers were completed the old concrete wall was removed directly in front of the piers so that timber struts could be placed between the trusses and the piers, thereby transferring the pressure of the top of the cofferdam from the old wall to the new piers. The old wall and the rock between the piers was then blasted and removed.

Excavation back of the cofferdam was carried down to a depth of about 6 to 8 ft. below the bottom of the cofferdam and within an average of about 2 ft. of the inside line of the cofferdam, and in a few places the rock broke back of this inside line. The concrete seal which had been placed in the bottom of the pockets could then be seen quite plainly in such places and showed that a good bond with the rock bottom had been made. As the excavation progressed downward and closer to the cofferdam, extra struts of 12 x 12-in. timbers were placed from the piers to the cofferdam. The view on page 699, which was taken back of the cofferdam, shows the piling of the cofferdam on the right, the piers on the left with the timber trusses above, and the excavation carried up to within about 2 ft. of the cofferdam. At the time when the excavation of the forebay was completed the entire leakage of the cofferdam and rock did not exceed 4 sec.-ft.

On page 698 is shown the completed cofferdam. As some of the piling drove into the bottom farther than was anticipated on account of there being more loose rock on the bottom than was expected, it was necessary to build a light wooden cofferdam along the top in order to take care of any extreme high water.

It was necessary to locate the power house as close to the cliff as possible in order to get the foundations and tailraces on solid rock. The top of the cliff here is about 218 ft. above the water level in the gorge. Extending upward about 100 ft. from the water surface is a talus slope which consists of a layer of about ten feet of loose rocks which had fallen from the cliff.

Excavation was begun at the toe of the slope using a ¾-yd. steam shovel, the loose rock being washed down to the shovel with a hydraulic giant. This hydraulic giant had a 4-in. nozzle and the water was supplied to it by a 10-in. pipe line carried from three of the 9-ft. penstocks in old Station No. 3. This gave a working head of about 180 ft. at the nozzle. As soon as sufficient area had been cleaned off with the hydraulic giant, drilling and blasting of the solid rock was begun. Excavation of the solid rock was carried down to about 4 ft. above mean water level. Then a concrete cofferdam was built on solid rock the length of the proposed power house and about 60 ft. west of the west wall of power house. Excavation was then carried down to 23 ft. below mean water level. The excavation above El. 370 consisted of shale and limestone, from this to the water surface alternate layers of Medina sandstone and red shale, and below the water level was solid Medina sandstone.

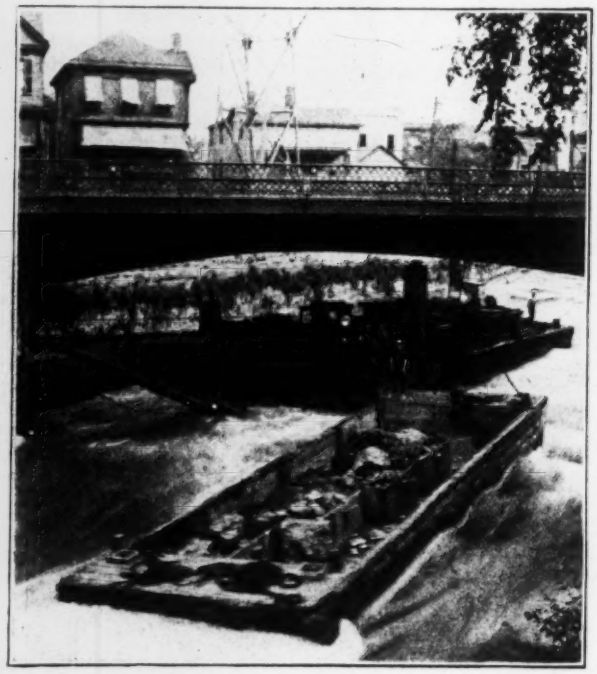


DREDGING IN A STIFF CURRENT IN THE NIAGARA HYDRAULIC CANAL

As soon as the excavation was completed the concrete foundations for the turbines were poured. The mixing plant was located at the top of the cliff. The gravel was brought in on cars and unloaded unto a belt conveyor which carried the gravel to a storage pile. It was then conveyed from the storage pile to hoppers over the mixers by a clamshell bucket operated from a derrick. The mixed concrete was conveyed through flexible sheet iron, elephant trunk, chutes suspended in cables extending from the top of the cliff to the bottom.

The piers and heavy reinforced-concrete floors for supporting the entire units were completed up to the bottom of the wheel cases. Then the wheel cases were set. After the cases were set the concrete piers and power-house floor were poured.

Manufacturers of all machinery were requested to ship in pieces not heavier than 50 tons, also that each piece must pass through a door 12 ft. wide and 16 ft. high.



LOW BOOM DREDGE FOR WORK UNDER BRIDGES

The equipment for the power house was carried below the bank by means of a large 50-ton crane which is a permanent structure built in 1906 and was used in the construction of old Station No. 3. This 50-ton crane is operated over a crane runway, the westerly end of which is a cantilever extending 53 ft. over the edge of the cliff and running back easterly over the switchtracks which are about 200 ft. from the edge of the cliff. The crane has a 50-ton and a 5-ton hook with a total lift of 235 ft. By means of this crane all equipment was lifted from the railroad cars upon the switchtracks, carried out to the end of the cantilever and lowered 225 ft. below unto a car which runs on a standard-gage track into the south end of old Station No. 3 and there turns into the new station. Within the new station are two electric cranes of 100 tons each. By means of these cranes all parts of the equipment were easily carried from the car and distributed to their proper place in the power house. In the erection of the turbines and generators it was necessary as the parts were assembled to make lifts from 150 to 200 tons. This was accomplished by using the two cranes together by means of a large whiffle tree, which was a 200-ton capacity box girder beam arranged so that the two cranes should be readily hitched at each end with a large lifting hook in the middle of this beam.

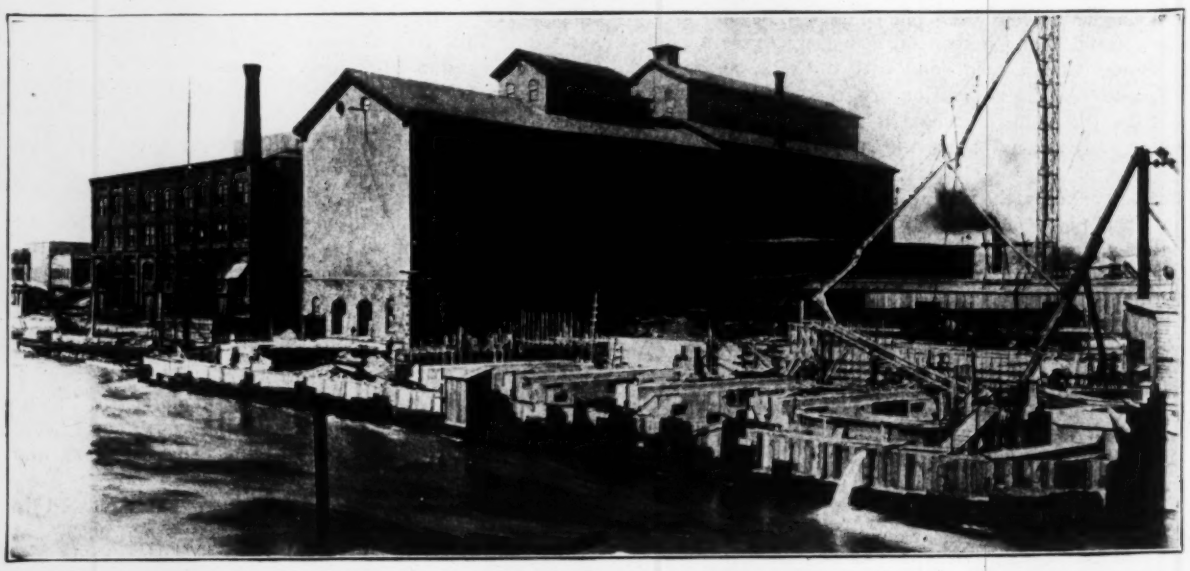
Tunneling and Lining the Penstocks

By G. W. Hewitt
Field Engineer, Niagara Falls Power Co., Niagara Falls, N. Y.

THE three penstocks of Station No. 3 extension are identical, and except for a few minor details during the construction, the conditions met were the same. Each tunnel was driven from the lower portal up through the rock to the top, a distance of 350 ft. Excavation started at the face of the deep excavation of the power house and ended at the face of the excavation of the forebay. The excavation of the bell-mouth section inside the limits of the forebay gatehouses was open excavation and was part of the excavation of the forebay.

The center line of tunnel was level at El. 353 to the P. C. of the lower curve, the P. I. of which was 86 ft. from the portal. At this point the line turned 18° 19′ to the right and 45° to the vertical, making the actual intersection angle 47° 50′. The two tangents were connected by a 40-ft. radius curve. The inclined section of tunnel was 45° to the vertical until the upper curve was reached, which also had a 40-ft. radius, and then into the open excavation of the bell-mouth section, the center line of which was El. 538, or a vertical distance of 185 ft. from center of lower horizontal section to horizontal section of bell-mouth.

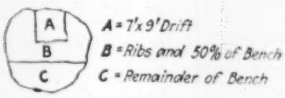
The diameter of finished tunnel is 15 ft. 6 in. and the plans called for a minimum of 18 in. of concrete, making the excavation 18 ft. 6 in. in diameter theo-



COFFERDAM AT FOREBAY OF EXTENSION TO STATION NO. 3, NIAGARA FALLS POWER CO.

retically, although sunflower cross-sections showed that the diameter of excavation averaged 20 ft.

Excavation was started as the lower portal, using four water Leyner column drills, and the entire heading, or upper half of the tunnel, drilled and shot. Disposal was made on the horizontal section by shoveling the rock into 4-yd. cars, which were hauled out by steam dinkeys. After the bench had been uncovered, it was then drilled and shot and the material disposed and the track run back. At the end of the horizontal section a heavy timber hopper was built under which the cars could be run and loaded so that after shooting the rock would roll down the 45° slope onto the hopper. Above this point and for the remainder of the tunnel, the drilling and shooting were done in three operations, as can be seen from the accompanying sketch.



STAGES IN PENSTOCK TUNNELING OPERATIONS

First a drift *A* approximately 7 ft. x 9 ft. was driven through to the upper portal, then the ribs and about 50 per cent of the bench *B*, and lastly the remainder of the bench *C*.

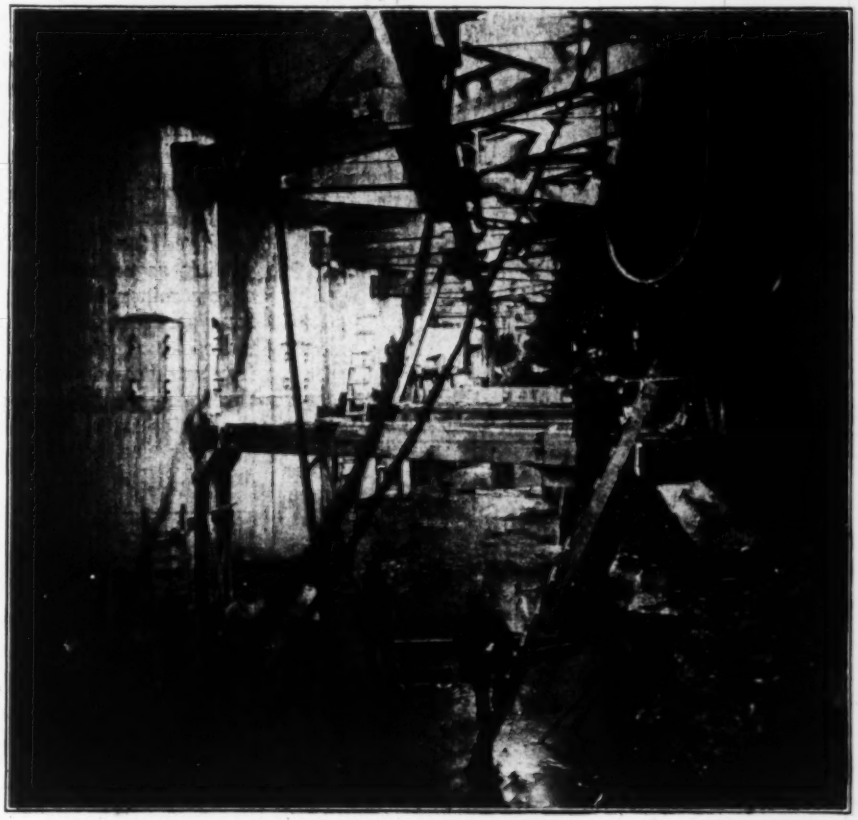
Excavation of the drift was naturally the most difficult operation, as the drilling and shooting of portions *B* and *C* more nearly approximated that of open excavation and were quite rapid. During excavation of the drift the work was carried on in two shifts of eleven hours each, working seven days a week. For the lower two-thirds of the 45° tunnel, each shift was able to drill and shoot a heading, averaging about 7 ft. each shot. Above that the progress slowed down owing to the difficulty of getting the heavy equipment of the 45° slope and also to the wet working conditions.

The drills, columns, and other equipment were hauled up the long slope by a rope passing through a block attached to an eyebolt in the roof. The planking and 8 x 8-in. timbers for staging were first hauled up and placed and the heading was then scaled, after which the columns and drills were hauled up and set up in place. The electricians and pipefitters went in as soon as possible after the rounds had been shot to make any repairs that had to be made to the lighting circuit and to the air and water lines.

After about 100 ft. up on the incline the air became so bad that an exhaust fan had to be installed and an 8-in. suction pipe attached to the roof of the tunnel. An auxiliary pump also had to be installed to keep up the water pressure.

The field party consisted of a chief, a transitman, and two chainmen. They gave two headings in each of the three tunnels every day as a rule, checked their lines and bases, estimated quantities, etc. As a rule they laid out the grade and roof line while the drillers were getting their material together. Grades as well as line were given with a transit using the vertical arc and correcting for distance of set-up above center line, which on a 45° slope was normal distance times the secant of 45° (1.414) equals the vertical distance.

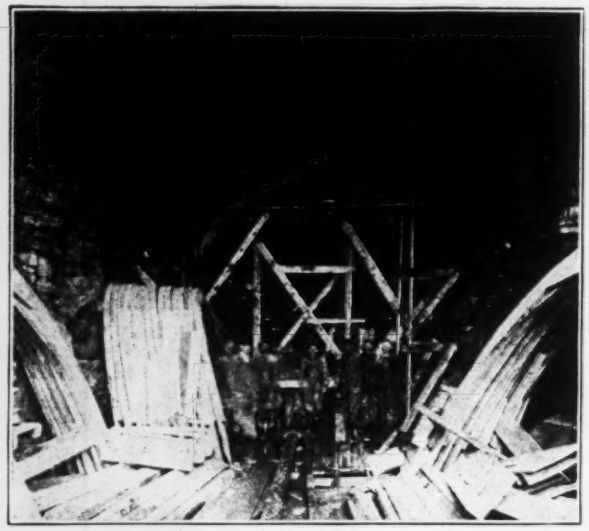
On the lower curve the horizontal and vertical projections were both parabolas owing to the circular curve being on an inclined plane. About five points on the curve were calculated for their projections and were then plotted up on cross-section paper and the curves were then drawn through them, prints made and furnished to the field party. In laying out the heading on the curve, the field party could always tell how far to the right and above the center of the horizontal section the center line was at the heading by picking the points of the curve without any long calculations of parabolic formulæ. They could also tell the heading foreman just how much he had to point his drills up for the next round of holes.



BEHIND THE FOREBAY COFFERDAM

The rock on the lower horizontal section was hard Medina sandstone. Above this, at the upper end of the curve, was a thin layer of shale overlaid by about 12 ft. of the Clinton ledge of limestone. A small amount of seepage was encountered in this shale in each of the three tunnels, while in the first one a drill opened up a water pocket which flowed for about twenty minutes and had a very strong sulphur smell. Above the Clinton limestone the tunnel went through approximately

100 ft. of shale, which was soft and stratified at the lower end and which shaded into an unstratified slate just under the limestone at the top. For the remainder of the distance limestone was encountered, varying from hard blue limestone just about the slate to well-seamed limestone near the top. The slate formation underneath the limestone was so hard that the drillers insisted that they were in the limestone heading before



TUNNEL HEADING OF PENSTOCK—OUTSIDE CUT ABOUT 20 FT. DIAMETER

they really reached it. The seepage in the upper part of the shale and lower portions of the limestone was such that it resembled a rainstorm, while considerable seepage was again encountered in the upper end.

The concreting of the lining was carried on in several independent operations in order to co-ordinate with the other work above and below the bank.

On the lower curve a 15-in. concrete invert section was first built with slots and pipe for the reinforcing. The wooden ribs were then set up and lined and then the wooden lagging placed. Two rows of circular reinforcing bars were then placed and the concreting done through chutes from the upper end of the tunnel. This section was the only one on which reinforcing bars were used.

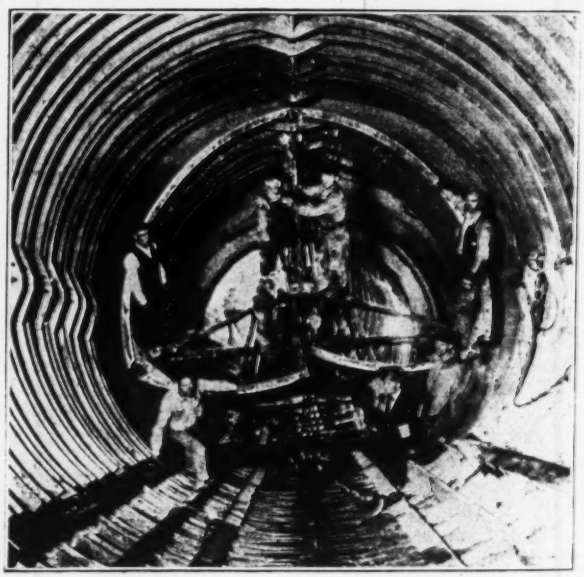
The seepage was taken care of by an 8-in. wrought-iron pipe emptying into the tailrace. A T was placed at the end of every 20-ft. section and a short length of 4-in. pipe and elbow attached to each side. Just below each elbow a dam was built out of sacks of concrete and just before the concrete reached the top of them they were covered with a piece of canvas and a few more sacks of concrete, thus leaving a permanent drain.

The bell-mouth section was concreted around wooden forms with a bulkhead between each tunnel. Concrete was carried to a height of 25 ft. above the bottom in a solid mass. The first pour was about a foot above the bottom so that the forms would not float, the second pour to slightly above center line, and the last pour to about 4 ft. above the highest part. The steel guides for the gates, anchor bolts for the racks, etc., were all set in place before concreting.

It was the original intention to concrete from the top of the lower curve to the top of underground portion with steel forms and then to connect the tunnel concrete with the bell-mouth section with wooden forms on the upper curve. However, as it was deemed advisable to rush the steelwork of the gatehouse in order to be able to use the crane, it was decided to concrete in the entire upper curve before doing that of the 45° slope. This necessitated making the joining at the top of the slope entirely underground. Pipes were placed along the roof of the curved section so that the slope below could be concreted at the connection.

On the 45° slope Blaw-Knox steel forms were used, having a diameter of 15 ft. 6 in. and a length of 5 ft. Twelve complete sections of forms and five extra invert sections were used. Concrete on this section was continuous from bottom to top, never being less than four batches per hour or just enough so that the top concrete had not set up when a new batch was poured. It was found that by having the men walk around and stirring up the top that so-called mud seams or seepage rings were prevented from forming. Forms were only moved during the day shift and iron workers were employed for this work. After the first two days in getting acquainted with their equipment and making a few minor changes in the traveler, they averaged four sections or 20 ft. each day. The traveler was moved by a hoist at the top. Besides the crew of five iron workers on the forms, there was a concrete crew of three men taking care of the concrete as it came through the chutes from the top.

Lastly the steel lining was placed and concreted around. The steel plates were assembled under the



FORM FOR CONCRETING PENSTOCK TUNNEL. CLEAR DIAMETER 15½ FT.

crane in the Johnson valve chamber and when riveted were skidded back and new sections attacked. This steel lining extended 68 ft. in from the edge of the cliff. The diameter was 15 ft. 6 in. except where connecting to the Johnson valve, at which point it was 17 ft.

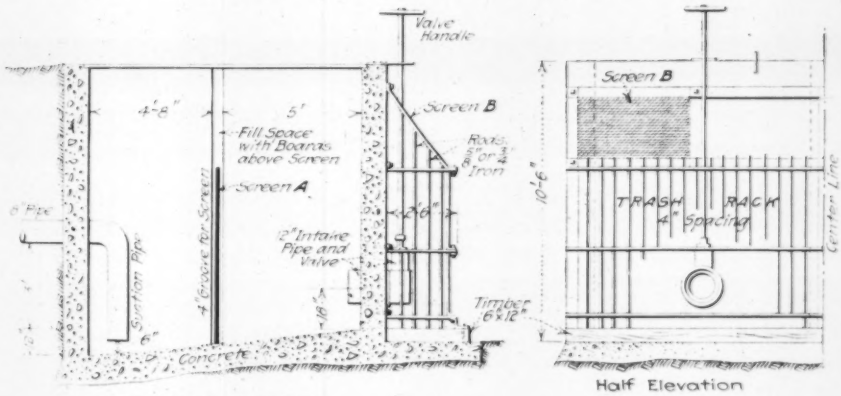
With the steel penstock in place the Johnson valve was next assembled and connected to the steel lining

and the wheel case. Riveting was then completed, burnt rivets being cut out and replaced by new ones. In the majority of cases, however, it was found that the electric weld on the rivet gave excellent results and made a tight job.

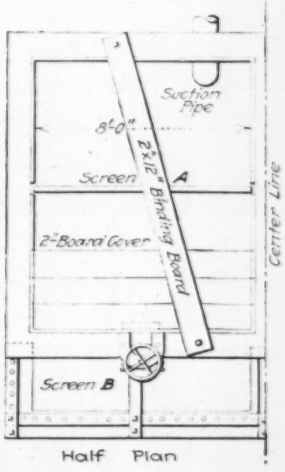
After the riveting was completed and all connections made, the portion under-ground was concreted in, grout pipes being placed along the roof through which grout was afterward forced under air pressure to fill any cavities along the roof which could not be concreted. At the lower end of the curve, where the joints in the concrete were made, a small keyway was left which was painted with soft asphaltum to prevent leakage in case of contraction. Several boxes were also placed at the joint with ¾-in. pipe leading from them into the open at the testing chamber so that any leakage from contraction at the joint would show.

Intake for Railroad Water Station Protected from Drift

A PROTECTED intake of simple design for the pumping plants of water stations has been built by the Baltimore & Ohio R.R. at several points where the supply is taken from streams which carry large quantities of drift. This intake consists of a chamber or suction well located at the edge of the bank and taking water through a pipe fitted with a valve. The design is used with either a single or double chamber, the latter



PROTECTED INTAKE FOR RAILWAY WATER STATIONS



of which is shown in the accompanying drawing.

Around the entrance of the intake pipe and valve is a protecting cage of rods, the cage being closed at the top by an inclined screen which excludes drift when freshets submerge the intake and prevents the drift from lodging as the water subsides. The intake pipe may be placed in either the front or side wall of the well and a second intake pipe at a higher level may be provided for use in case of obstruction of the lower pipe, but this precaution has not been found necessary in present installations. The intake chamber is divided by a removable screen sliding in guides, the suction pipe from the pump entering the rear of the chamber. Netting for the spark arresters of locomotives may be utilized for the screens.

This form of intake was designed by S. O. Benjamin, 1323 Hollins St., Baltimore, Md. while he was in the water service department of the Baltimore & Ohio Railroad.

Pumping Project with Low-Level Storage and Semi-Gravity Supply

By R. A. Trufant
Engineer, Llano Grande Plantation, Mercedes, Texas

IN ONE of the lower Rio Grande valley newest pumping projects, precedent was largely disregarded. The settling and storage basin is located below the average ground level, eliminating seepage. Gates are provided for filling this low level reservoir by gravity during rises of the river, and pumps to fill the reservoir during low stages of the river. The river storage pump and the main canal supply pump are duplicate 30-in. electrically-driven units using power from a full-Diesel oil engine and a 115-kw. generator. The overall efficiency of the plant is figured at 20 per cent against the 4 to 6 per cent common in the valley. The gravity supply features further reduces operating costs to an extent not yet determined.

At this point the Rio Grande is bordered by lakes formed by the meandering of the river. The common practice is to build a levee around a lake up to the main canal level and to use the basin thus formed for storage and settling. Thus seepage is away from storage, and the maximum lift is about 25 ft. On the Llano Grande project, the lakes are to be held at a level midway between lower river level and the main canal. Seepage is then toward the storage. Two pumps are necessary, one to lift the water from the river into storage and one from storage into the canals. But these pumps (and the power plant to operate them) need be only half the size of those ordinarily employed, as the maximum lift is halved. The river pump is to be operated nights and the main canal pump days when needed for irrigation.

The river is subject to frequent short rises; therefore gates are provided between the river and the lakes, so that the storage can be filled by gravity in half a day from the crest of the rise. Thus the river pump need be operated only during protracted periods of low river.

The cost of installation, about $100,000, is somewhat higher than for the average local plant, but the economies of operation will more than compensate for this added cost. The plant is designed to irrigate one-third of the 7,000-acre Llano Grande Plantation

near Mercedes, Tex., owned by W. M. Bancroft and planted largely to sugar cane.

The equipment was furnished by the Allis-Chalmers Co., the construction work was done by Dodds and Wedegartner, of San Benito, contractors, and the design was made and supervision handled by the writer as engineer of the plantation.

Curves of Building Cost Increases Used in Valuation Work

By F. E. Barnes
Supervising Building Valuation Engineer, New York Central Lines

As is well known to most engineers, the Interstate Commerce Commission has been making a physical valuation of all railroads of the United States. To do this it was necessary to make a field inventory of all railroad property, which was so large an undertaking that it was impossible to complete the work in any one year. It therefore became necessary to give each railroad a valuation date, and various roads were assigned different years, ranging from 1914 to 1920. All roads are, however, being valued by using prices which were in effect in 1914 in order that the results would be comparable.

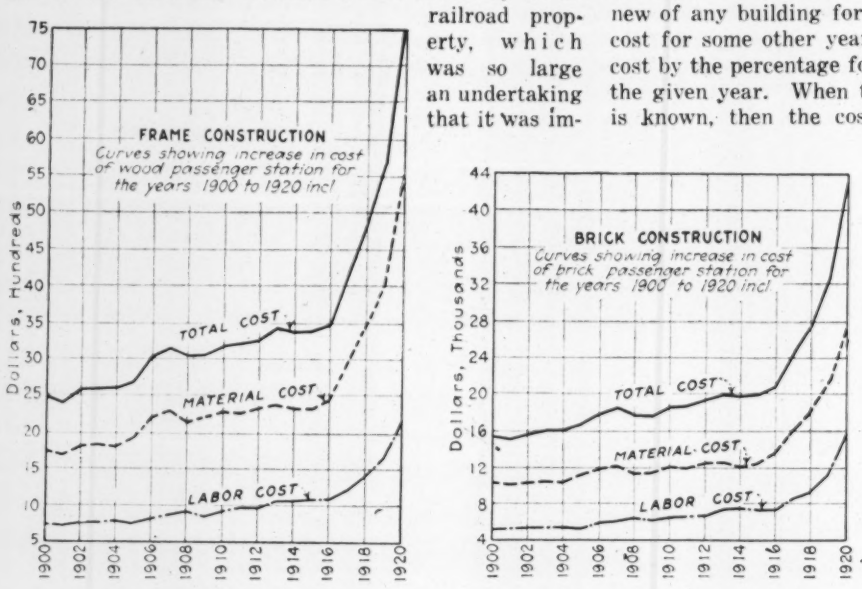

FRAME CONSTRUCTION
Curves showing increase in cost of wood passenger station for the years 1900 to 1920 incl
Dollars, Hundreds
TOTAL COST
MATERIAL COST
LABOR COST
BRICK CONSTRUCTION
Curves showing increase in cost of brick passenger station for the years 1900 to 1920 incl
Dollars, Thousands
TOTAL COST
MATERIAL COST
LABOR COST

FIG. 1. INCREASE IN BUILDING COSTS FOR FRAME AND BRICK STRUCTURES

Although the railroad records show the construction costs of many of their buildings, yet these costs are seldom for the year 1914. The writer, therefore, has prepared cost trend curves by the use of which the probable cost of a building in 1914 can be obtained when its original cost and date of construction are known. For this purpose bills of material were prepared for typical buildings, both frame and masonry, keeping the labor costs separate from the material costs. These bills of material and labor were then priced for each year from 1900 to 1920 inclusive and the results plotted on a curve as shown in Fig. 1. The labor cost, material cost and the total cost are shown separately.

Fig. 2, which shows the relation of cost between any two years, was prepared as follows: The cost of the building for the year 1914 was taken as 100 per cent and the cost for any other year was shown as a percentage of the 1914 cost. This is shown by the curve made with a dotted line and designated the percentage curve. The curve shown with the full line is the reciprocal of the percentage curve, and is simply one divided by the percentage. By the use of these two curves it is possible, when the cost of a building for any given year is known, to determine what the cost would be for any other year covered by the diagram. This can be done as follows: To find the equivalent cost of construction new of any building for the year 1914 when the actual cost for some other year is known, multiply the actual cost by the percentage found on the reciprocal curve for the given year. When the cost new for the year 1914 is known, then the cost for any other year can be found by multiplying the cost for the year 1914 by the percentage found on the percentage curve for the year in question.

The curves as shown on the diagrams given were prepared from bills of material for a standard frame passenger station 22 ft. wide by 55 ft. long and for a brick passenger station 31 ft. wide by 133 ft. long. The prices of material and labor were for Albany, N. Y. These curves would not necessarily be accurate for other class of buildings nor for the same buildings in other parts of the country, but by actual test the writer has concluded that the curves in all cases will be very similar to those given, and will not vary for different classes of buildings, or even different sections of the country, as much as ordinarily would be expected for different types of construction and locations.

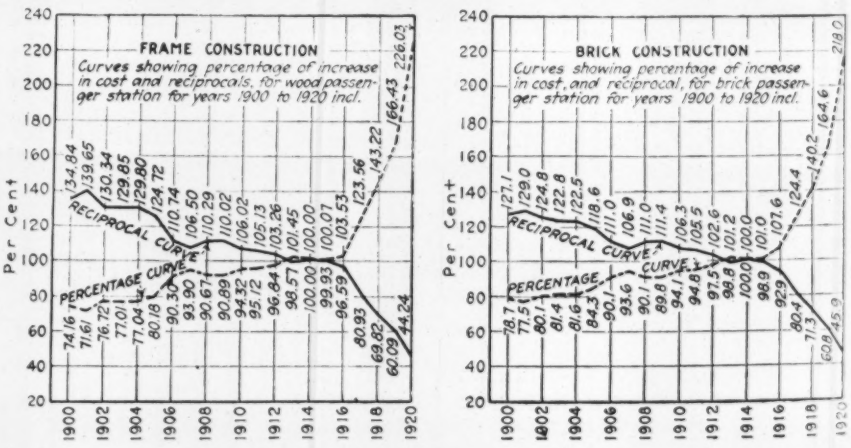

FRAME CONSTRUCTION
Curves showing percentage of increase in cost and reciprocals, for wood passenger station for years 1900 to 1920 incl.
Per Cent
RECIPROCAL CURVE
PERCENTAGE CURVE
BRICK CONSTRUCTION
Curves showing percentage of increase in cost, and reciprocal, for brick passenger station for years 1900 to 1920 incl.
Per Cent
RECIPROCAL CURVE
PERCENTAGE CURVE

FIG. 2. RECIPROCALS OF BUILDING COST INCREASES
Based on 1914 prices and used in railroad valuation.

Such curves are useful not only in valuation work, but if kept constantly up to date for determining the proper fire insurance schedules, probable cost to construct similar buildings, etc.

City-Manager Plan Widely Endorsed

TO SECURE first-hand information on the practicability of a city-manager form of government, the Sacramento Chamber of Commerce made inquiry of Chambers of Commerce, newspaper editors and individual citizens in a number of cities where the plan is in operation. Such letters were not addressed to city managers or other city officials because an unprejudiced opinion was sought.

In the letters of response there was a sweeping expression in favor of the city-manager plan, according to *Pacific Municipalities*, in which typical replies are published. There was not a single instance in which the plan was declared to be a failure but there were many enthusiastic endorsements of it. Other letters sent to cities having the commission form of government brought "a large majority of answers voicing dissatisfaction and complaints, a number stating that there was strong sentiment for the city-manager form." The accompanying list gives the names of cities with an estimated population of 20,000 or more now operating under or pledged to some form of city-manager government. The total number of all cities of the United States so operating on March 25, 1920, was 177.

State	City	Pop.	In Effect	City Manager Salary	City Manager Vocation
Ariz.	Phoenix	40,000	April, 1914	$5,000	Mech. Engr.
Cal.	San Diego	95,000	May, 1915	4,000	
	San Jose	40,000	July, 1916	6,000	Physician
	Santa Barbara	20,000	Jan., 1918	4,000	
Iowa	Dubuque	47,500	April, 1920		
Kan.	Wichita	75,000	April, 1917	6,000	Elect. Engr.
Mich.	Grand Rapids	165,000	Mar., 1917	5,000	Banker
	Jackson	52,000	Jan., 1915	4,000	Civil Engr.
	Kalamazoo	55,000	June, 1918	6,000	Sec. Chamb. Com.
	Muskegon	50,000	Jan., 1920		
N. M.	Albuquerque	20,000	Jan., 1918	5,000	Civil Engr.
N. Y.	Auburn	40,000	Jan., 1920	4,000	
	Newburgh	30,000	Jan., 1916	3,600	Business
	Niagara Falls	55,000	Jan., 1916	5,000	Civil Engr.
	Watertown	40,000	Jan., 1920	7,500	Civil Engr.
N. C.	Gastonia	20,000	Aug., 1919	3,600	
Ohio	Akron	200,000	Jan., 1920	10,000	Attorney
	Ashtabula	21,500	Jan., 1916	3,000	Business
	Dayton	170,000	Jan., 1914	7,500	Civil Engr.
	E. Cleveland	25,000	Jan., 1918	6,000	Civil Engr.
	Sandusky	25,000	Jan., 1916	5,000	Business
	Springfield	70,000	Jan., 1914	6,000	Civil Engr.
Okla.	Muskogee	50,000	April, 1920		
Penn.	Altoona	65,000	Jan., 1918	7,500	Civil Engr.
Texas	Amarillo	20,000	Dec., 1912	2,500	Business
	Beaumont	35,000	May, 1920		
	Ranger	30,000	May, 1919		
Va.	Lynchburg	35,000	June, 1920		
	Newport News	37,500	June, 1920		
	Norfolk	115,777	Sept., 1918	12,000	Civil Engr.
	Petersburg	25,000	Sept., 1920		
	Portsmouth	80,000	Jan., 1917	5,000	Civil Engr.
	Roanoke	47,000	Sept. 1918	4,800	Civil Engr.
W. Va.	Charleston	43,000	May, 1915	4,500	
	Wheeling	80,000	July, 1917	8,000	

Cleveland, Ohio, population 500,000, will vote on City Manager Charter this fall.

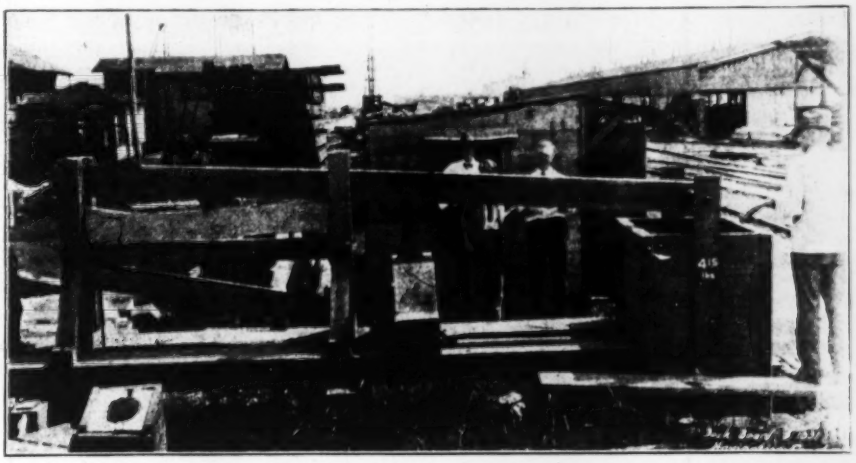
Control Cement Content by Field Beam Tests

Machine Devised on New Orleans Inner Harbor Canal Which Measures Breaking Load of Large Concrete Beams

By Marshall G. Findley

Assistant Designing Engineer, Board of Commissioners of the Port of New Orleans, New Orleans, La.

IN THE design of the reinforced-concrete lock for the New Orleans Inner Harbor Navigation Canal, considerations of weight and cost led to the use of unit stresses somewhat higher than usual. As a protection, however, it was specified that the concrete should test in compression on 8 x 16 in. cylinders at least 2,200 lb. per square inch at the end of 28 days. In the early part of the work these cylinders were made at New Orleans and shipped to the laboratory of Columbia University, in New York City, for test, but delays in transit led to the establishment on the work of a home-made testing machine, in which the center breaking load on a concrete beam could be measured. This machine is now being used to control the proportions of the concrete.



TESTING THE CONCRETE BEAM ON THE JOB

In March, 1919, 106 cylinders were cast and sent to New York. These varied in mix and water content and showed that while the water would account for certain variations in strength, there were apparently differences due to the quality of the cement, which was being stored on the site in large quantities. It was felt, therefore, that there should be some check on the cement and its proportion in the mix, so the following system was established:

The cement storehouse was divided into sections and each individual section filled as far as possible from one shipment so as to have, if possible, uniform quality throughout. When filled, representative samples were selected and six cylinders made using a 1 : 2 : 4 mix and as dry a mixture as could be placed in the lock. After two weeks these samples were shipped to the Columbia testing laboratory and tested. On receiving results of these tests in New Orleans a decision as to proper proportions could be made, using 1 : 2 : 4 concrete when the cement tested well and 1 : 1½ : 3 concrete when the strength showed below 2,000 lb. per square inch.

This procedure seemed to meet all demands for a

COMPUTED UNIT STRESS IN OUTER FIBER OF BEAMS BROKEN IN MACHINE, MAY-JUNE, 1920

Concrete = 1 : 2 : 4
Age = 15 days

Failure = Compression, as a rule, in lb. per sq.in.
Each figure represents one beam.

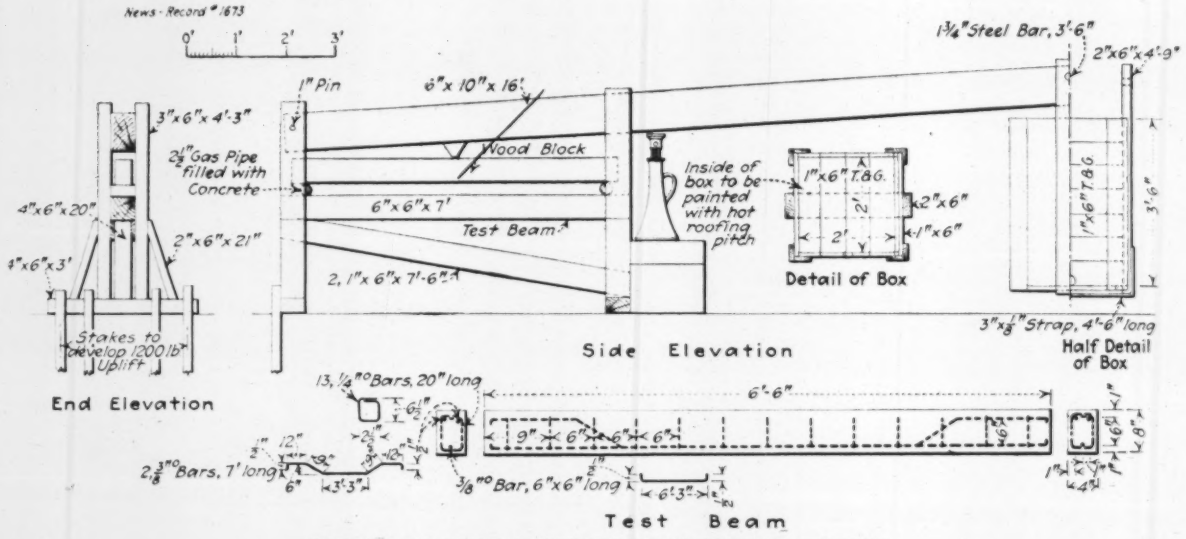
Sec. 3 Lost 1B, 1A	Sec. 3 Lot 1C	Sec. 7 Lot 2	Sec. 2 Lot 1	Sec. 6 Lot 2	Sec. 1 Lot 1	Sec. 5 Lot 2	Sec. 4 Lot 2
2,100	1,760	2,130	2,050	2,680	1,790	2,210	2,470
2,285	1,970	2,230	2,185	2,470	1,635	2,525	2,525
1,945	2,130	2,310	2,000	2,500	1,580	2,105	2,470
2,285	2,130	2,180	1,920	2,260	1,820	2,365	2,315
2,390	2,075	2,270	2,155	2,420	1,820	2,315	2,365
2,285	2,075	2,260	2,210	2,470	1,710	2,470	2,155
Av. 2,215	2,023	2,230	2,087	2,467	1,726	2,332	2,382

time, when transportation difficulties began to make trouble of two kinds. The first of these was that cement was being used faster than it was received, and it became evident that even if all else went well, it would soon be necessary to use cement from sections on which a report had not yet been received. Reports on cylinders were in general received by telegraph about six weeks after the cylinders were made. Then, in addition, it was found that the cylinders, although sent by express, were being delayed between New Orleans and New York. Orders were then given to attempt a method of locally testing the concrete in beams, based on the results of a short previous investigation.

Previous to this the suggestion had been made that tests of small beams might be as valuable as tests of cylinders. On account of the fact that cylinder tests are better standardized, it was not intended that beam tests should take the place of cylinder tests. But nevertheless a machine was designed, as shown in the drawings. This consists of a heavy wood frame lever machine, in which the load, a tank of water, is applied at the center of a standard 4 x 8-in. by 6 ft. 6 in. concrete beam, the breaking load being readily measured by the inches of water in the tank. In the tests reported in the accompanying table, for instance, the weights and leverage are such that the total load on the center of the bear $W = 2{,}578 + 155\ h$ where h = depth in inches of the water in the loading tank. Using the parabolic formula, which applies when the beam is at or near failure, and with constants determined for the test beam the compression fiber stress $f_c = 870 + 52.4\ h$. Thus for a depth of water of 23½ in. as for the first result noted in the table, the value of $f_c = 2{,}100$ lb. per square inch.

Some preliminary tests carried out on beams 7, 14 and 35 days old were not altogether satisfactory on account of variation in results, so the method was laid aside temporarily.

In May, 1920, it became evident that reports of cylinder tests could not be received in time for use and it was decided to try the beam tests again, using specimens 15 days old, which was just about the leeway possible. The accompanying table shows the results of these first tests. They were quite consistent and satisfactorily high, so that it was thought safe to use a mix approximately 1:2:4 for all the cements except those from



DETAILS OF FIELD CONCRETE BEAM TESTING MACHINE

Section 1, for which the proportion of cement was increased by 16⅔ per cent. Attention might be called to the fact that figured by the straight-line formula, it would take about one-quarter of the average breaking load on these beams to stress the concrete in compression to 800 lb. per square inch, the extreme figure allowed in design.

The above investigations were carried on under the direction of the consulting engineers, represented in New Orleans at first by George M. Wells and afterward by Col. George R. Goethals. The cylinder tests were made by Prof. A. H. Beyer, of Columbia University, and under the direction of Modjeski and Angier, testing engineers.

Chicago Bascule Bridge

In the article on the Michigan Ave. double-deck bascule bridge, in *Engineering News-Record* of Sept. 9, p. 508, an intended correction of the matter on safety devices was omitted. As the installation of these devices and the signal system has not been completed it should be stated that this equipment "has been designed" instead of "was installed."

Sheet Asphalt Maintenance Costs in District of Columbia

Use of Salvaged Material Makes Cost $1.74 Square Yard, Including Asphalt Cost, Haulage, Cutting Out, Working Patch

By Major F. S. Besson
Assistant Engineer Commissioner, District of Columbia

THE EFFECT of the severe winter was, during the early spring months of this year, very noticeable on a goodly portion of the 170 mi., comprising 3,334,157 sq.yd., of sheet asphalt pavements in the District of Columbia. It is our general experience that streets surfaced with sheet asphalt need no repairs until after they reach an average age of seven years, and the effect of the severe conditions instanced was immaterial on pavements within the age limit, but on older streets, it was most apparent. As sheet asphalt pavements grow older they require an increasing amount of attention which cannot be interrupted if the streets involved are to be kept in condition above criticism.

During a usual Washington winter, the highway division is able to keep its sheet asphalt repair gangs at work more or less continuously. The past winter was an exception and for several months work had to be entirely discontinued. Ordinarily, from an economic standpoint, defective spots are repaired while in what might be called an incipient state. Instead, during the winter, while covered with snow or slush and subjected to heavy traffic, the defective spots could not be repaired, and developed into bad holes, daily becoming larger and larger and more numerous. As soon as weather conditions permitted in the Spring, four repair gangs were organized, averaging twenty men to the gang. By July 1, the end of the fiscal year, each had averaged 61 working days, and our sheet asphalt streets were in first class condition. Ninety three thousand cubic feet of asphaltic material were used. More than 35,000 separate patches were made, the averaged sized patch being 0.9 sq.yd., laid at an actual cost of $2.07 per square yard.

Asphalt Plant Operation

The normal output of the municipal asphalt plant is but 800 cu.ft. per day, therefore it had to be supplemented by purchases from a local contractor having the contract for new pavement. The material purchased was standard asphalt binder and topping, 38c. per cubic foot being paid for the former and 48c. per cubic foot for the latter. The coarse aggregate for the mixture turned out by the municipal plant consisted of old material salvaged from previous cuts and resurfacing jobs. The average cost of the hot product was but 25c. per cubic foot. Based on the use of old material alone, the cost per square yard for patching was $1.74.

There are several points of interest in the operation of the municipal asphalt plant. The old material is broken to a maximum 1½-in. size by means of a crusher consisting of a box 2 ft. x 2 ft. x 2 ft., with a floor made of horizontal 2-in. square steel grate bars. Within the box are revolved a number of hammer heads attached to a shaft by chain link handles. The hammer heads are of flat steel, 4 in. x 8 in. x 1¼ in., the striking edge being the 8-in. x 1¼-in. face. The shaft makes approximately 1,000 revolutions per minute, and the hammer heads strike upon and break up the old material, as brought from the streets. This is fed into the top of the box, the finely broken product escaping through the bottom bars. The cost of crushing is $1.40 per cubic yard. Usually no crushing is done during the warm summer months, as it has been found better to handle and crush old material during cool weather. From the crusher the product is delivered to store piles from where it may be taken with the aid of a pick even after a storage of several months.

The asphalt plant is a portable one purchased in 1912. Its batch capacity is normally 24 cu.ft. of hot mixture. The process with old material in the plant is generally similar to usual practice with all new materials. Prior to reaching the mixing chamber, it is passed through a drying drum, a charge for which consists of 15.6 cu.ft. of old material, 8.4 cu.ft. of sand and 75 lb. of limestone dust. The drum is operated by steam power and fuel oil furnishes a hot blast through it to dry and heat the charge. Each Sunday, because of the asphaltic content of the old material, it is necessary to put a man inside the drum to clean off the accumulation sticking to the sides. From the drying drum the charge is shifted into the mixing chamber which is adjoining it on the same shaft. To the charge in this chamber is added 100 lb. of asphaltic cement from a melting kettle, the heat for which is furnished by a wood fire. This item of asphaltic cement presents one of the significant economies in the use of old material which itself is given credit as having a 100-lb. asphaltic content per charge. In other words, with the addition of 100 lb. of new cement, a hot mixture is obtained equivalent to a 200-lb. asphaltic content mixture wherein new materials are used for the aggregate.

Manufacturing Cost

The cost of a cubic foot of old material mixture is as follows:

0.65 cu.ft. of old material	$0.0337
0.35 cu.ft of sand	0.0273
3.12 lb. limestone dust	0.0069
4.16 lb. asphaltic cement	0.0426
Total cost of material, 1 cu.ft. of hot mixture	$0.1105

Hauling to the plant is included in the above costs, except in the case of the old material for which the hauling is charged to the street work from which it is salvaged. Ten per cent is added to the list price of asphaltic cement in order to make an allowance for tare.

Cost of Salvaged Material

The gang for plant operation consists of one overseer, one steam engineer, one skilled laborer, one kettle man, one drum man, one elevator man, five material men, and one utility man. Manufacturing cost per cubic foot is as follows:

Labor	$0.1211
Plant maintenance	0.0198
(b) Total manufacturing cost	$0.1409

No charge is made for plant amortization since it has reached an age where such is no longer necessary. In fact, very little of the original plant remains, it having been replaced bit by bit in running repairs. Neither is a charge made in connection with the buildings and ground occupied.

The cost of old material mixture at the plant is therefore:

(a)	Material for 1 cu.ft.	$0.1105
(b)	Manufacturing 1 cu.ft.	0.1409
(c)	Cost of mixture 1 cu.ft.	$0.2514

The completed hot mixture is discharged directly into trucks fitted with dump bodies, for transportation to the streets. These trucks are hired by the District and are of two types: One carries two batches or 48 cu.ft., the other carries one batch. During the past season's work the average distances from the contractor's plant and from the municipal plant to the job on the street were approximately equal. The length of haul may be judged by the fact that five was the average number of loads per truck per day for the 93,000 cu.ft. of hot mixture delivered. (d) The bill for hauling totalled $7,037.30, or 7½c. per cubic foot.

It has been ascertained that there are 3 cu.ft. of hot mixture per square yard of area repaired, based upon which the material cost delivered on the job, in the case of the old material output of the municipal plant, is as follows:

(c)	Material for 3 cu.ft. of hot mixture	$0.7542
(d)	Hauling, 3 cu.ft.	0.2250
	Total cost, per square yard, of material delivered on the job	$0.9792

The standard asphalt pavement in Washington is sheet asphalt with a binder course of 2 in. before compression and a topping specified as 2½ in. before compression, which gives a finished pavement of approximately 3½ in., laid on a 6-in. concrete base. When a rough, or otherwise bad spot appears in a pavement, the sheet asphalt in the defective area is cut out down to the concrete base. The cost for cutting out, painting the edges of the cut with hot asphaltic cement, maintaining equipment and tools, and hauling the old material from the streets amounts to $0.3905 per square yard of area cut out.

The cost of spreading the hot mixture in the patches, rolling and dusting the surface with portland cement, totals $0.3729 per square yard. Three of the patching gangs worked with hand rollers weighing approximately 1,500 lb. and operated by three laborers. The fourth gang used a small steam roller weighing 2½ tons. The results obtained, both as to costs and quality of work done, did not differ materially for either method of rolling.

Adding the 37.29c. per square yard for working the patch, to the 39.05c. per square yard for the cutting out, gives a total cost for the repair work on the streets of 76.34c. per square yard.

Of the aforementioned $1.74 per square yard for patching with old material, hot mixture is seen to be comprised of two costs: 98c. for the material delivered to the job; and 76c. for the actual work on the street.

During the short Spring working season up to July 1, the end of the fiscal year 1920, $65,000 was spent on patching, which is a maintenance rate of 2c. per square yard for the entire yardage in place. Previously during the fiscal year, $55,000 had been spent for maintenance, giving a total of $120,000 for the entire year, or a rate of 3.64c. per square yard. This figure may be taken as representing the maintenance on a group of average pavements under present traffic, labor, and material conditions. It has long been considered that Washington sheet asphalt pavements with a 6-in. concrete base give 25 years of service before requiring replacement of the asphaltic surface. During the early years of this service the maintenance is practically nothing while as the pavement approaches its age limit the maintenance rapidly rises, but with conditions as at present it may be taken that the average for each year of its life of 25 years, approximates slightly more than 3c. per square yard. For some pavements the rate is below this and for others above. Some are resurfaced long before giving 25 years of service, while other existing Washington asphalt pavements have reached an age of 40 years.

In making a study of defective pavements, it is seen that some surfaces crack excessively, and others, while they do not crack, roll into bumps and depressions, causing uncomfortable and slow travel over them. The four repair gangs in the District have been making patches at the rate of more than 500 per day. A close scrutiny of this work shows that cracks in the asphaltic surface bear absolutely no relation to whatever cracks there may be in the concrete base. Also that surface rolling is independent of whether the concrete base presents a smooth or rough foundation. Cracking and rolling are dependent upon the skill used in selecting the materials and handling them in the construction of the pavement. It is practicable to obtain a surface that will neither crack nor roll. In trying to obtain this ideal it is considered that defects due to cracking can more readily be repaired to meet the requirements of traffic than can those due to rolling.

Streets Needing Replacement

Of much importance is the bearing that the amount of money spent for repairs has relative to a demand for resurfacing. Is maintenance the only essential in determining when replacement of the asphaltic surface shall take place? From a survey of the District there has been prepared a list of asphalt streets now needing resurfacing. This list comprises 141 items, well distributed throughout the District and averaging between 2,000 and 2,500 sq.yd. each. The total is 330,025 sq.yd., which, at a cost of $4.75 per square yard for stripping the old surfacing, making incidental adjustments and placing a new surface, amounts to practically $1,500,000. The age of the pavements in question range up to 46 years with an average of 29. The cost of repairs per square yard for these streets each year for the past seven, averages slightly less than 4½c. With conditions as at present it may be estimated that this figure for the coming twelve months will approximate 6c., and this is the only tangible expenditure for which data are available for comparison with the yearly amortization charges that would be necessitated if a new surface were to be substituted.

Charges for a replacement to cost $4.75 per square yard and to have an estimated life of 25 years with money at 5 per cent are:

23.75	cents per square yard per year for interest
2.50	cents per square yard equated annual repairs
9.50	cents per square yard depreciation annuity
Total 35.75	cents per square yard amortization.

In other words, it costs six times as much to resurface the pavements as it does to continue to repair them. In making a decision therefore, in favor of resurfacing it

is evident that there are elements for consideration other than that of maintenance, and of five times the importance. At present these elements are rather intangible, comprising as they do the comfort of passenger traffic, waste of time used in traveling, appearance of the pavement from the viewpoint of civic pride, and increase in ton-mile operating cost due to rough highway surfaces. The necessity of placing any particular street on the resurfacing list was determined not by an examination of the street's data card in the office, but by a study of its actual physical condition. Similarly the order of urgency on the list was determined not by relative maintenance costs but by relative conditions of the surfaces presented.

In contrast to asphalt pavements in their prime, the surfaces presented by the streets listed for resurfacing are rather rough. Furthermore, on these worn out streets it is not practicable to raise the expenditures for repairs to such a point as to make the rough surfaces the equal of new. The only remedy is to resurface. Rough pavements mean increased maintenance and depreciation expenses for motor vehicles. Many truck owners do not understand costs of operation, and while sincere in the belief that they are making a comfortable living, are really using up their original investment for current expense. The measure of the actual efficiency of a vehicle is the engineer's ton-mile cost as determined by the gross weight transported including the tare of the machine. If, because of these rough pavements the engineer's ton-mile cost is raised but ½c., it amounts to an excess charge for the traffic of 1,000,000 tons per mile per year, using these streets, to $5,000. For a street of 20,000 sq.yd. to the mile this signifies 25c. per square yard, a charge relatively large in comparison with that of but 6c. per square yard for road repairs.

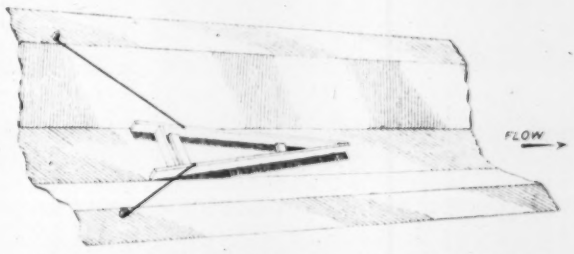
Principles Governing Replacement

Highway funds for the District are dependent upon appropriations made by Congress and that body reflects the attitude of the general public which is rather indifferent to slight increases in fuel consumption and intangible depreciation. Interference with the comfort of the travelling public is a great, and more often than not, the sole influence in directing attention to a poor highway, but this influence usually aids one or two favorite thoroughfares to the neglect of many others equally deserving. Not one of the items, (1) increase in maintenance, (2) comfort of passenger traffic, (3) waste of time used in travelling, (4) appearance of the pavement from the viewpoint of civic pride, or (5) increase in ton-mile operating cost; standing by itself, is sufficient to warrant resurfacing, and the highway engineer is the only one who gives relative merit to all in order to arrive at a rational conclusion. A great number of pavements may be listed for resurfacing, but until such time as the public is educated to appreciate the elements upon which the list is based, appropriations are going to be sufficient to accomplish but a very small portion of the list, and this is borne out by Washington experience.

For a decade or more it has been considered that Washington's asphalt streets give an average service of twenty-five years. To accept this as true to date and to maintain this status with 3,334,157 sq.yd. of pavement in place, 120,000 sq.yd. of resurfacing should be accomplished this year, and the amount should increase 2 per cent yearly in order to keep pace with additional new construction. Such an ideal has not been achieved in the past. If it had the average age of all Washington asphaltic pavements would be 12½ years. To the contrary, there are 670,000 sq.yd. beyond 25 years of age, and the average age is close to 16 years. Of late years, instead of increasing, the resurfacing yardage accomplished each year has been decreasing. Though 330,000 sq.yd. with an average age of 29 years may be listed for resurfacing, appropriations are sufficient to carry out less than 50,000 sq.yd. For the future it must be considered that instead of 25 years, 30 years of service are rendered by the average Washington sheet asphalt pavement. Instead of a synthesis of engineering principles governing, the real criterion is "Satisfy the Public."

Sand Removed From Ditch By Scouring

TO CLEAR irrigation ditches of drifted sand a scouring device has been used with success on the Mapleton district of the Strawberry Valley project, in Utah. As shown in the accompanying drawing this device consists of a timber A-frame, which is placed in the canal with its nose pointing downstream and is kept in position by guy lines attached to the upstream end and either secured to stakes or held by men on the banks.



FRAME FOR SCOURING DRIFT SAND FROM CANAL

When the frame is loaded with rock so that it rests on the bottom, the water rushing under it scours out the sand. The frame settles until the desired depth is attained. Then the lines are released and the frame is allowed to move slowly downstream, the water scouring out the sand ahead of it.

In the Mapleton canal nearly 15 in. of drift sand was removed in this way in 1919. The method is described in the *Reclamation Record* of August, 1920, by T. R. Smith, hydrographer on the Strawberry Valley project of the U. S. Reclamation Service. A somewhat similar scouring method on the Umatilla project, but with the use of a vertical template shaped to the cross section of the canal or ditch, was described in *Engineering News-Record* of Dec. 11, 1919, p. 998.

Noted Irrigation Project Completed

Completion of the North Side Twin Falls Irrigation Project has been announced. This is one of the earliest and for a long time most discouraging projects in Idaho which takes water from the Snake River. It is now one of the most prosperous of the Carey Act enterprises, has been accepted as completed by the state and turned over to the settlers for management. Work of segregation began in 1906. Of the 170,000 acres involved 145,000 are now under cultivation. The canal system is 800 miles long and the total cost of construction is $4,500,000.

Notes from Foreign Fields

SWITZERLAND'S SPECIAL ENGINEERING PROBLEMS—THE ENGINEER'S PART IN EXPLOITING SWISS SCENERY

By E. J. Mehren
Editor, *Engineering News-Record*

PHYSICAL conditions give the engineers of Switzerland a concentration of difficult and interesting transportation problems. The country is mountainous, the passes few and steep, the valleys tortuous. The provision of transportation facilities under these conditions is difficult and expensive.

Moreover, the population is relatively dense, while on every side are countries normally busy and wealthy. As a result there is heavy through as well as heavy internal traffic.

The picturesque character of the country adds to the transportation problem, since the tourist wants to go in comfort to places that for commercial or agricultural reasons might well be served with quite primitive transportation agencies.

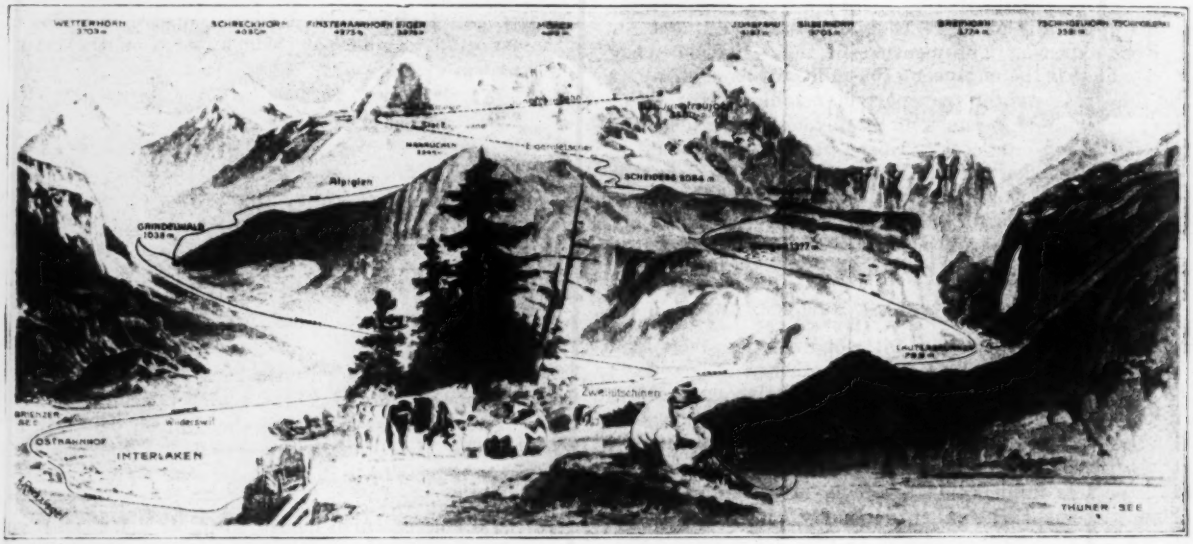
The engineering visitor, therefore, can well afford to spend much time in studying the Swiss railways, the highways, the tunnels and the bridges.

Almost of equal interest are the stream-control and the water-power problems. The streams from the snowclad mountains are obvious sources of water power, while the concentration of population makes it necessary to use every square foot of cultivable ground and to safeguard it from the destructive action of the swift mountain watercourses. The art of mountain-stream control and the regulation of rivers have, accordingly, attained a high development, and offer a set of interesting solutions to those who can find time to study them.

The main line railways are, in general, of the same gage as our own, constructed as a rule with maximum curves of 300-m. radius (somewhat less than 6-deg. curve). The grades run to very high figures, some as high as 6.6 per cent. On the heavy through roads, such as those connecting with the St. Gotthard and the Simplon tunnels, the grades are held to 2.7 per cent.

There is also a large mileage of meter-gage road, principally through difficult mountain country. On these meter-gage roads the maximum curvature is, as a rule, of 120-m. radius (14 deg. 38 min.), though on the famous Chur-Arosa line the maximum curve has a radius of 60 m. (about 29 deg.). The grades on the adhesion sections are as high as 7 per cent, but on some of the rack-rail sections, principally on mountain scenic lines, the grades run to 25 per cent, and in one case, the Pilatus Railway, to 48 per cent.

The chief railway tunnels of Switzerland are known by name to every engineer. The Simplon, he knows, is 12.3 miles long, the St. Gotthard (double-tracked) 9.3 miles, but he is probably unprepared for the statement that there are no less than ten other tunnels which have lengths greater than 3½ miles. The Loetschberg is 9 miles long, the Mont Cenis 8 miles, and the Arlberg 6.3 miles, all three being double-tracked.



MOST FAMOUS OF SWISS MOUNTAIN-CLIMBING RAILROADS AND ITS CONNECTIONS
The Jungfrau Railway starts at Scheidegg, near the center of the picture.

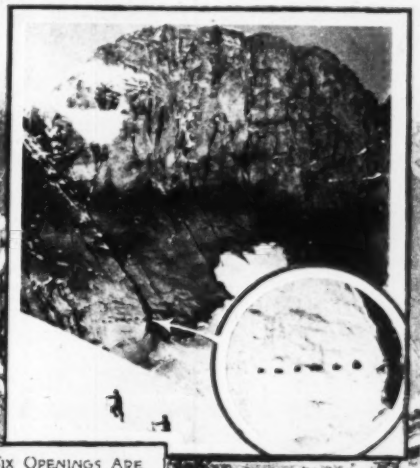
Naturally the highways present interesting problems. The traffic, as a rule, is light, with relatively few automobiles, so that waterbound macadam, laid both with and without large-stone bases, has been principally used, though for the last 30 years large-stone foundations have been considered standard on the important roads. Tar macadam and bituminous spraying are coming into larger use. Improvement work, however, is hampered now since the costs are three times what they were before the war.

The principal roads have widths of from 5 to 7 m. and, in the mountains, maximum grades of about 10 per cent. On roads in the high passes short stretches of 12-per cent grade are found.

The development to keep the grades down to the 10 per cent maximum in the mountains is very elaborate. A view herewith shows a section of the famous

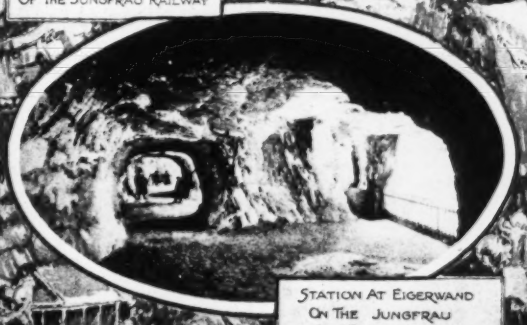
FOUR PAGES of SWISS ENGINEERING ACHIEVMENTS

ILLUSTRATING ALSO, THE ENGINEER'S PART IN EXPLOITING SWISS SCENERY

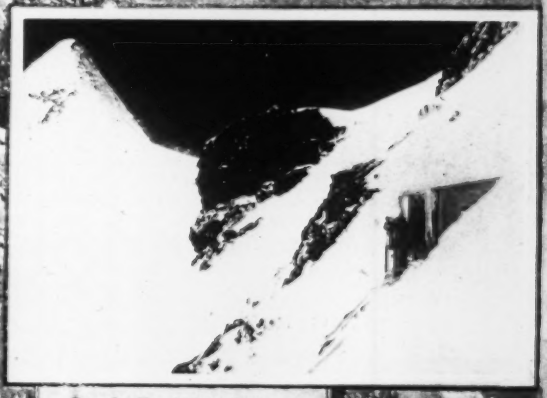


THE SIX OPENINGS ARE THE EISMEER STATION OF THE JUNGFRAU RAILWAY

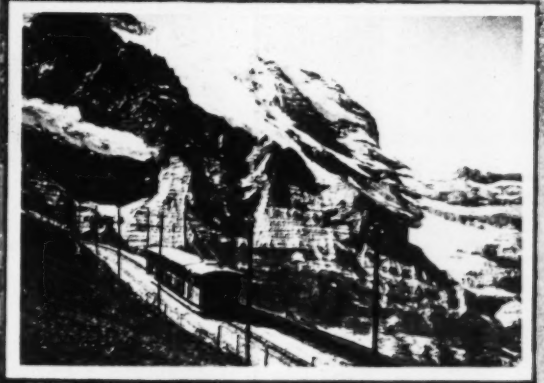
HIGHEST RAILWAY STATION IN EUROPE — JUNGFRAUJOCH 11,340 FEET ABOVE SEA LEVEL



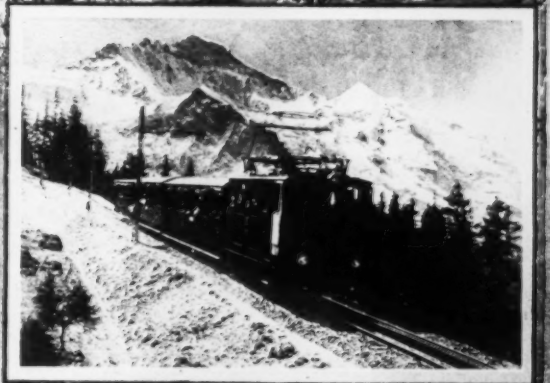
STATION AT EIGERWAND ON THE JUNGFRAU RAILWAY



ONE OF THE VIEWS FROM THE JUNGFRAUJOCH STATION



NEAR THE STARTING POINT JUNGFRAU RAILWAY — MOUNTAIN IN THE FOREGROUND IS THE SILBERHORN ONE OF THE JUNGFRAU GROUP



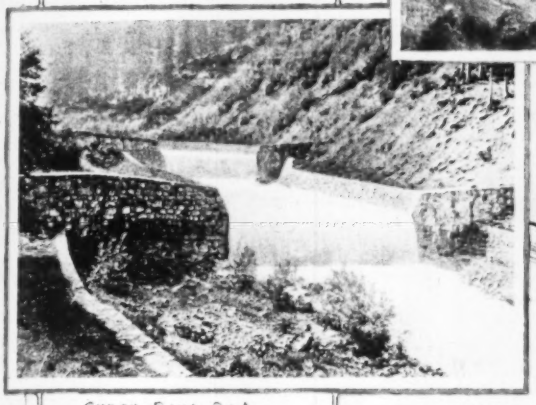
JUNGFRAU — AN EARLY WINTER PICTURE — CHARACTER OF VIEWS FROM MOST FAMOUS OF SWISS MOUNTAIN RAILWAYS

PHOTOS BY PHOTOGLOB-ZURICH

ROCK GALLERY ON THE AXEN ROAD — ONE OF THE MOST FAMOUS HIGHWAYS OF THE MOUNTAIN DISTRICT
STEEP SECTION ON THE RACK RAILROAD ASCENDING MT. PILATUS
EHGE
EINSANG
BÜRGENSTOCK LIFT
ON THE LAKE of LUCERNE
ELEVATOR RISES 537 FT.
WETTERHORN LIFT
AERIAL CABLEWAY AT GRINDELWALD — WITH RISE OF 1370 FEET
ON THE ROAD FROM WEESEN TO AMDEN NEAR LAKE WALLEN
PHOTOS BY PHOTOGLOB, ZURICH

HIGHWAY OVER THE
KLAUSSEN PASS
A FAMOUS SWISS ROAD
ELEVATION OF SUMMIT 6396 FT.

DAM PROTECTION
ON THE RHONE
AT A POINT HALF A MILE FROM THE
NORTH PORTAL OF THE SIMPLON TUNNEL



CHECK DAMS ON A
MOUNTAIN STREAM—
TORRENT CONTROL HAS BEEN
HIGHLY DEVELOPED IN SWITZERLAND

ELECTRIC LOCOMOTIVE
ON MOUNTAIN RAILWAY—
OPERATED BY RACK SYSTEM
ON THE STEEPER GRADES

AERIAL CABLEWAY
AT GRINDELWALD
ANOTHER VIEW OF THIS INSTALLATION SHOWN ON A PRECEEDING PAGE

LOWER REACHES OF A
MOUNTAIN STREAM—
THE SCHAECHEN-CHECK DAMS
ARE USED ON THE UPPER REACHES.

SWISS SOLUTIONS of BRIDGING PROBLEMS
GRÜNJETOBEL BRIDGE
ON THE CHUR-AROSA RAILWAY
SPAN – 275½ FT.
WIESEN VIADUCT
ON THE RHAETIAN RAILWAY–
CENTER SPAN 180 FEET
KORNHAUS BRIDGE
A FINE HIGHWAY AND ELECTRIC
RAILWAY BRIDGE WITH A CENTRAL
SPAN OF 350 FT.
LANGWIESER VIADUCT
ON THE CHUR–AROSA RAILWAY–
LONGEST REINFORCED CONCRETE
SPAN IN SWITZERLAND 321½ FT.
LANDWASSER VIADUCT
ON THE ALBULA RAILWAY –
BUILT ON CURVE OF 328 FOOT RADIUS
PHOTOS BY PHOTOGLOB ZURICH

Klaussen Strasse, which goes over the Klaussen Pass from Altdorf to Linthal. The maximum elevation attained is 1,952 m. (6,402 ft.), and of this height the last 3,122 ft. are attained in a distance of 7½ miles.

Bridges—A few photographs herewith, beneath each of which will be found the span of the particular structure, gives some idea of the character of the solutions worked out by the Swiss engineers for their bridging problems.

Engineers' Part in Exploiting Swiss Scenery

What has been said so far of the transportation facilities (except as to the Klaussen Strasse) applies chiefly to lines of communication essential for commercial traffic. There is another system of communication whose sole object is to carry tourists to points otherwise attainable only through difficult mountain climbing. In other words, the engineer has lent his genius to the exploitation of Swiss scenery, and the resulting work form in themselves a most fascinating study.

The most notable, by far, of the mountain railways is the Jungfraubahn (the Jungfrau Railway). The art of mountain-railroad building had been highly developed in Switzerland by the beginning of the last decade of the Nineteenth Century. There were cable inclines and rack railroads in large number. Many of the latter had been built under great difficulties and were noteworthy examples of engineering. For the most part, though, they were below altitudes of 6,000 ft., and were free from snow the greater part of the year. Moreover, traversing the lower slopes, they did not have to contend with sheer precipices, and deep gorges, even though on most of them grades of 25 per cent were necessary. None of the roads offered precedent for surmounting the difficulties of the higher regions—eternal snow, glaciers and precipitous cliffs.

Nevertheless, a bold spirit dreamed of plans for conquering by engineering means the Jungfrau, the most famous and possibly the grandest, even if not the highest, mountain of Switzerland. Realizing the futility of trying to build a line on the face of the mountain, understandable even from the pictures shown herewith, he conceived the idea of tunneling to the top and of breaking out observation stations at points affording magnificent views. Fortunately, financial backing was available, and in 1897 the project worked out by Adolph Guyer-Zeller was put under way.

The plan contemplated a line 7½ miles long, starting at Kleine Sheidegg, a station on the Wengernalp Railway, at an elevation of 6,770 ft., and ascending by rack railroad to a point inside the mountain, at an elevation of 13,430 ft., directly under the Jungfrau summit. From this point an elevator was to rise through a height of 240 ft., placing the tourist without physical effort on the tip of the Jungfrau, 13,670 ft. above the sea. A station was to be established in the open at Eigergletscher, 1¼ miles from the starting point, but from Mile 2 the line was to be entirely in tunnel, three observation stations being broken out between Eigergletscher and the summit.

The project has been carried out for a length of 5¾ miles, to Jungfraujoch, at an elevation of 11,340 ft. The final stretch to the summit has not been built.

The line is of meter gage, with a single-track tunnel, 12 ft. wide and 14.3 ft. high. Operation is under adhesion up to grades of 6.6 per cent. Above that the rack system, with maximum gradient of 25 per cent, is used. The trains are electrically driven, with current supplied from near-by hydro-electric stations. The cost of construction was 12,000,000 francs (about $2,400,000 at normal exchange rates), and the highest section, from Eismeer to Jungfraujoch was put in operation in 1912.

The start is made along a length of adhesion track affording a magnificent view of the great mountain mass. At Eigergletscher opportunity for studying a large glacier is afforded, and after a short stop the train soon plunges into the long tunnel. The speed is low, 11 miles per hour on the adhesion and 5½ miles on the rack sections, but the one-and-a-half-mile stage to Eigerwand station is traversed before one has finished musing on the fine scene that has just been left behind. The station is a long arched room, with two tracks and two platforms, from which a passage, 26 ft. long, leads to a gallery parallel with, and just inside of, the face of the mountain. Along the gallery are three drifts, each 18 ft. wide, to the face of the mountain, affording a fine view of the valleys to the north. The passage, the valley and the drifts are, of course, all excavated out of the solid rock.

From Eigerwand, at the north face of the mountain, the line crosses to the south, and at Eismeer presents a magnificent picture—a glacier basin, surrounded by the high peaks of the Barnese Range. From the engineering point-of-view the Eismeer station is the most remarkable on the line. As at Eigerwand, a passage leads from the platforms to the face of the mountain, but instead of merely a bare rock gallery, there is here a finely-finished and appointed dining room, with capacity for seating 200 guests at one time. The cooking and heating are done by electricity. The six wide windows of this station, as seen from the outside, is shown in one of the accompanying views.

The final climb is at the 25 per cent gradient. The present terminus, Jungfraujoch, at elevation 11,340, has a small restaurant, a post-office, and sleeping rooms for those who want to see sunset and sunrise from this point of vantage or make mountain-climbing tours. The Jungfrau, it is said, is easy of mastery from this point, the ascent, over snow and ice even in midsummer, requiring 3½ hours. The view here is unsurpassed, some idea of what can be seen being afforded by one of the illustrations.

As a piece of engineering the line deserves a visit no less than for the magnificent panorama it affords. A quick ride to the top of the world is necessarily an interesting experience, particularly to an engineer, who appreciates the toil required to build this comfortable mountain-climbing railroad. One does not even suffer from the cold, for the cars, like the stations, are electrically heated—a really necessary provision, for out on the balcony at Jungfraujoch and in the gallery and drifts at Eigerwand one quickly became chilled to the bone even in July. Mountain climbers look with contempt on the weaklings who do their peak-scaling by railroad, but for the average mortal the Jungfrau Railway affords the only means of getting to the top of a peak of eternal snow.

For that boon the engineer is to be thanked, and particularly Guyer-Zeller, a tablet in whose honor now looks out on the panorama of the Eigerwand station.

Other Mountain Lines—While the Jungfrau line is

by all odds the most interesting of the mountain railways, there are others that would deserve mention if space allowed. The grades, as a rule, are 25 per cent maximum, but a very notable exception is that of the Pilatus Railway, which has maximum grades of 48 per cent. A photograph of one of the steep sections is shown herewith.

Both steam and electric power are used for driving these mountain lines. On the steam lines the braking on the rack sections is effected through bands on both sides of the pinions which engage the rack. On the electric roads both this type of braking is employed and also electric braking—by running the motors as generators, working against a resistance.

There are many cable roads, but they are of relatively short length. There is also one cableway installation—the famous Wetterhorn Lift, illustrated herewith. At this installation two cars operated so as to counterweight each other travel on cables and rise through a distance of 1,376 ft. above the Grindelwald Valley.

Another installation of unusual character is a high-lift elevator at Buergenstock, illustrated herewith. It stands on the face of an almost sheer cliff and has a total travel of 537 feet.

Yet another means of communication developed by engineering science is coming into use in Switzerland, though with it the civil engineer has relatively little to do. The company has instituted an aëroplane service on four of the Swiss lakes: For 50 francs one can have a 15 minutes' flight, whilst special flights from lake to lake can be arranged to suit the tourist. At present machines are in service, indicating either successful commercial operation or the faith of the promoters in the development of the enterprise.

Mountain Stream Control

Reference has been made in a previous article to present progress in the development of Swiss water powers, so that further reference is not necessary here. A word should be said, however, with reference to the very extensive operations in the control of mountain streams. These naturally bring down large quantities of debris and if uncontrolled deepen their beds and create constant havoc.

In the upper reaches the building of small dams, of the type illustrated in the accompanying photographs, is the common remedy. These have the same purpose, of breaking the flow and thus catching the debris, as the dams built experimentally by Mr. Olmstead in the mountains near Los Angeles, but the construction is more substantial.

In the lower reaches dry-laid and mortar-laid bank protection is used, while with the larger streams, after the floor of the valley is reached, spur dykes are in common use. Along the Rhone, at Brigue, where the north portal of the Simplon Tunnel is located, the banks are protected by dry walls and spur dykes 100 ft. apart. A picture of these dykes is shown herewith.

In the 50 years ending in 1913 no less than 139,000,000 francs were spent on stream correction and control in the lower reaches and 56,000,000 francs in the higher reaches.

Under the control of the same branch of the government which looks after the streams is the study and prevention of land and snow slides. Work is constantly under way at points where dangerous conditions are developing, the method being to intercept the water which is saturating a seam and creating a slippage plane.

Conclusion.—These problems and solutions offer to the American engineer a variety of interesting studies that tempts one to spend fully as much time in this small country as in any of its larger neighbors. In the larger countries one can study conditions and solutions which are akin to and have application in our own country. In Switzerland it is rather the novelty of conditions that attracts.

Zurich, July, 9.

Harbor Piloting and Ship Location by Radio Methods

THE PROBLEM of piloting vessels by radio signal has been studied from several points of view recently. The Navy Department is just completing a very important series of tests in the New York Harbor on a system using what is called a "radio piloting cable." This system was first tried out by the Navy Department nearly a year ago but the first tests were not successful due to damage of the cable by fishing operations. A new cable designed by the radio division of the Bureau of Engineering of the Navy Department has, however, proved successful and recent tests are reported to be exceedingly encouraging.

The work requires a cable energized by a low frequency alternating current laid along the center of the channel through which vessels are to be guided. The magnetic waves from this cable are picked up by any one of several types of receiving equipment aboard ship and the pilot lays his course in accordance with the relative intensity of the signals from the right or the left receiving units. A full report upon this work will be available on completion of the present tests on best intensity and character of signals, most suitable type of receiving equipment, and other important technical details.

In connection with the radio location of ships the Navy has now under consideration the establishment of a large number of additional radio compass stations. These will be placed along the Western seaboard and on the Great Lakes. It is anticipated that they will find extensive use in directing the merchant marine, though they will be designed primarily for naval service.

Another problem also under investigation by the Government is being studied through the Bureau of Navigation of the Department of Commerce. This investigation relates to determination of ship location by radio signal. The system in use at the present time by the Navy employs radio compass stations on shore which measure the angle from which the signal is sent by the vessel to be located. By a system of triangulation two or more shore stations working together can determine the location of a vessel. The new system under test by the Department of Commerce reverses the operations by sending the signals from shore stations and doing the radio compass work aboard ship. Recent tests on this system are reported to have been very successful, but further investigations both as to apparatus and methods of manipulation are anticipated before any extensive application will be recommended.

Economic Weight of Rail for Varying Traffic Conditions

ECONOMIC VALUE of rails of different weights in relation to cost of rail maintenance is determined by the traffic, and a minimum traffic of 900 cars per track per day is the economic limit between rails of 100 lb. and 130 lb. per yard. These statements are the principle conclusions of a report presented at the recent annual meeting of the Roadmasters and Maintenance of Way Association by J. B. Baker, engineer of maintenance of way, Pennsylvania System, Cleveland.

Records for seven stretches of 100-lb. rails show an actual life ranging from 14 to 24 years and a traffic of 400 to 900 cars per track per day. On seven stretches having 917 to 1,700 cars per track per day the life ranged from 5½ to 20 years, but in two of these records the life was only 5.5 and 8.1 years. The number of cars per track per day as the unit of measure for traffic conditions was adopted as representing closely the locomotive tons and gross tons, and because the figure at any point can be determined without difficulty.

RELATIVE LIFE AND ECONOMY OF RAILS OF DIFFERENT WEIGHTS

Cars per Track per day	100-lb. Rail		130-lb. Rail		85-lb. Rail	
	Life Years	Annual Cost Per Mile	Life Years*	Annual Maint. Cost Per Mile	Life Years†	Annual Maint. Cost Per Mile
1,600	10.30	$1,250	20.6	$1,175	5.89	$1,400
1,400	11.40	1,200	22.8	1,145	6.52	1,400
1,200	12.75	1,140	22.5	1,100	7.29	1,320
1,000	14.10	1,090	28.2	1,080	8.06	1,250
800	15.90	1,040	31.8	1,050	9.09	1,165
600	18.40	990	.....	.....	10.50	1,095
400	21.10	940	.....	.....	12.05	1,025

*Life of 130-lb. double that of 100-lb. rail.
†Life of 85-lb. ⅘ that of 100-lb. rail.

A heavier rail is economical, as a general proposition, when its longer life will make the annual cost less than that for a lighter rail. The annual labor cost of rail maintenance will become less as the weight of rail is increased, less labor being required to install and maintain the heavier rail under nearly all conditions. The design of track may be changed radically by increasing the weight of rail and decreasing the number of ties, but this is a question of economic track design. There is also a special case where a very heavy rail will give practically infinite life and at the same time make possible a very low annual labor charge for maintenance. This point is illustrated where electric railways of light traffic use rails of a weight and strength which bear no relation to the traffic conditions, the economy of the track structure being due to the very long life and the small cost of maintenance.

For a uniform life of ten years, it is estimated that the annual cost of maintenance per mile of track is $1,116, $1,269 and $1,587 for 85-, 100- and 130-lb. rails respectively. But it is estimated that under uniform traffic conditions the life of the 100-lb. rail will be 1.75 that of the 85-lb., and the life of the 130-lb. will be double that of the 100-lb. or 3.5 times that of the 85 lb. He gives the average life at 6.55, 8.25 and 12.85 years for the three weights respectively. A comparison of life and cost under different traffic conditions is given in the above table.

On the basis of relative life, the report states that on track where an 85-lb. rail lasts only two years and has an annual maintenance cost of $3.327 per mile of track, the life for 100-lb. and 130-lb. rails would be 3½ and 7 years respectively, with corresponding maintenance costs of $2,490 and $1,945. Under these conditions the 130-lb. rail is the most economical. Similarly, where an 85-lb. rail has a life of nine years and an annual maintenance cost of $1,176, the life for the two heavier rails will be 15¾ and 31½ years, with maintenance costs of $1,030 and $1,050 respectively. In this case the 100-lb. rail is the most economical.

Land Classification Used by Farm-Loan Engineer Appraisers

Water Supply, Construction Service and Drainage Conditions Factors on Which Farm Loan Bureau Approves Loans

ENGINEERING factors in irrigated farm appraisals as utilized by the engineer-appraisers of the Federal Farm Loan Bureau was the subject of an article in *Engineering News-Record*, June 3, 1920, p. 1100. That article gave four definite classifications for the adequacy of water supply. The bearing of construction and drainage conditions on loan value was also discussed but no classifications were attempted. J. T. Whistler, engineer adviser to the Federal Farm Loan Board, has recently set up classifications for (1) what is termed construction service and (2) drainage conditions to cover the remaining factors. By construction service is meant character of service furnished by the constructed irrigation system in which generally cost of operation and maintenance is the principal factor and continuity of service a secondary factor. Mr. Whistler's classification of construction service follows:

Class A—Construction service in which the actual or estimated average annual cost of operation and maintenance per acre, plus the estimated annual hazard of loss per acre through break in service, does not exceed 5 per cent of the average annual mean value per acre of the three principal or staple crops of the district, on the basis of Class I water supply. By mean value per acre of the three principal crops is meant the total value in the district of the three crops divided by the total acreage in the three crops.

Class B—Construction service in which the cost of operation and maintenance, plus hazard of loss, exceeds 5 per cent but does not exceed 10 per cent, as defined for Class A.

Class C—Construction service in which the cost of operation and maintenance, plus hazard of loss, exceeds 10 per cent but does not exceed 20 per cent, as defined for Class A.

Class D—Construction service in which the cost of operation and maintenance, plus hazard of loss, exceeds 20 per cent, as defined for Class A.

Construction classification for applications in considering loans covers not only good or poor construction, as ordinarily considered, but also operation and maintenance of construction. A high operation and maintenance cost may be directly the result of poor design, but is often entirely due to natural conditions which make low operation and maintenance cost impossible, however good the design may be. For this reason the construction factor was combined with that of operation and maintenance and denominated construction service.

The relative weight that these factors bear to each other in the character of construction service will vary with conditions: (1) The degree of need for irrigation, that is, rainfall available for crops and it

variation from normal; (2) character of crops—for example, the loss from water failure one season might not only lose an entire season's yield from an orchard or from alfalfa but might also result in loss of the orchard or of the alfalfa stand itself; (3) value of crops and dependence of farmers on irrigation of lands; (4) cost of construction as compared to value of land; (5) cost of normal operation and maintenance.

A replacement cost, as of a wood-stave pipe line, wood flume, or other structure of a perishable nature, might be considered to constitute another factor. But it is held by Mr. Whistler that such cost is properly a matter affecting operation and maintenance (except as it may affect also the factor of continuity of service), and that it should be distributed to operation and maintenance cost. Similarly, the possibility of failure of a dam, for example, from faulty design or insufficient spillway capacity is not covered by either factor, but it will affect operation and maintenance cost and may materially affect the factor of assurance of continuity of service.

The following illustration is given: In a system involving little or no hazard of failure, operation and maintenance may have a relative weight of from 75 to 100 per cent. In a system depending wholly or largely on storage, in which the failure of a dam might possibly result in abandonment of the project, the assurance of continuity of service may have a relative weight of from 50 to 100 per cent. In a system depending on a pipe line comparatively inexpensive to replace but which (because of location or inaccessibility) cannot be replaced during the season, the factor of continuity of service may have a relative weight of from 10 to 20 per cent or even greater, depending on the value of crops which would be lost and plants destroyed.

Three principal or staple crops, instead of one as originally proposed, are used as the basis for computing the classification, since it was realized that there should be included not only the principal farming crops of forage and grain but also one or more of the intensive crops, in order that the labor conditions which so affect land values might be brought out.

The relative value of the different classes of construction service will vary with conditions, such as (1) labor supply, affecting crops grown, (2) climatic conditions, both as to temperature and as to rainfall, and (3) transportation and market facilities. This variation in value is true also of the water-supply classification. In a general way, however, there is an irrigation value of irrigated land separate and more or less distinct from what may be termed the "dry land" value. In a region of low humidity and little rainfall, as in most of the Southwest and the larger part of the Great Basin, the irrigation value is for all practical purposes the entire value. In regions of high humidity and of from 10 to 20 in. of rainfall during or available in the growing season, the irrigation value is less.

Classes of Drainage Conditions

Definitions for drainage conditions are necessarily more complicated because there are two distinct kinds of drainage troubles. One is due to alkalification, the other to simple drowning of the plants. Alfalfa, clover, timothy and grains may thrive on land where the ground water is within 2 ft. of the surface, but some lands with ground water 5 ft. from the surface may be absolutely barren. The four classes for drainage conditions adapted by Mr. Whistler are as follows:

Class "Good"—Drainage conditions are "Good" where permanent natural drainage or satisfactorily operating artificial drainage (or both) are now and will continue such that there will be no harmful accumulation of alkaline salts on or near the surface of the ground through capillarity, and such that the soil is and will continue otherwise so drained that there will be no loss therefrom in excess of 5 per cent of the average annual mean value per acre of the three principal or staple crops grown or adapted to the district, on the basis of Class I water supply and Class A construction service.

Class "Fair"—Drainage conditions are "Fair" where they would be in class "Good" except that adverse conditions are and will continue such that the crop value as defined in class "Good" is reduced more than 5 per cent but not more than 20 per cent. Drainage conditions are also "Fair" where they would come within class "Good" as defined but where the ground water is slowly rising not more than 1 in. per year, provided that the water table at the present time is not closer than 8 ft. from the surface at any time of the year, that the district is well established and has practiced artificial drainage successfully for several years and provided that it is estimated the land may be drained, if it becomes necessary, at a reasonable cost.

Class "Poor"—Drainage conditions are "Poor" where they would be in class "Good" except that adverse conditions are and will continue such that the crop value as defined in class "Good" is reduced more than 20 per cent but not more than 50 per cent, or such that the same effect is produced by limiting the choice of crops to those adapted to insufficiently drained soils, such as the short-rooted crops and "wild hay" and "salt grasses." Drainage conditions are also "Poor" where they would otherwise come within class "Good" at the present time, but where the ground water is slowly rising not more than 2 in. a year, provided that the water table at the present time is not closer than 6 ft. from the surface at any time of the year, that the district is well established and has practiced artificial drainage successfully for several years and it is estimated that the land may be drained, as it becomes necessary, at a reasonable cost.

Class "Bad"—Drainage conditions are "Bad" where the mean crop value as defined under class "Good" is reduced more than 50 per cent on account of the adverse drainage conditions, where the ground water is closer than 6 ft. from the surface and is slowly rising, where the ground water though lower than 6 ft. but not lower than 8 ft. from the surface is rising at the rate of more than 2 in. a year, or where it is probable that the development of adjacent or nearby land will within the period of the loan bring about such conditions.

The definitions undertake to provide not only for drainage conditions as affected by alkalification but also as affected by high or threatened high-water table and waterlogging, where there may be no excessive alkalies either in the soil or in the water. Even where objectionable alkalies exist either in the soil or in the water to a harmful extent, conditions as to water supply for leaching, or as to character of soil permitting it, may materially modify otherwise adverse conditions and change the classification. The amount of rainfall is an important factor since with from 15 to 20 in. of rainfall on average soils objectionable alkalies are likely to be leached out naturally.

Water-Supply Classification

Class I—Water supplies which have no shortage during ordinary years.

Class II—Water supplies which have some shortage in normal years, but which furnish sufficient water to mature crops. Where storage is involved, those providing two-thirds or more of a full supply are in this class if not in Class I.

Class III—Water supplies which furnish water only during the normal flood season. Where storage is involved, those providing one-third to two-thirds of a full supply.

Class IV—Water supplies which furnish during the irrigating season a full supply for not to exceed 30 days in normal years and none in years of low run-off. Winter irrigation supplies and those providing less than one-third of a full supply where storage is involved.

The engineer-appraisers are primarily employed as engineers but they also are required to appraise lands. It has developed that they are called upon by the banks to approve or otherwise comment on all irrigated land appraisals before loans are finally passed by the banks. The Farm Loan Board now requires engineer-appraiser reports on all irrigated lands with the exception of certain defined small areas that have been irrigated for ten years or more, and in which no change is likely to occur before the securities will be approved for bond issue.

LETTERS TO THE EDITOR

The Wall Street Explosion

Sir—Much has been printed in the daily press regarding the Wall St. explosion and two things are conspicuous: The immediate conclusion of the federal authorities that the Reds were at work, and the alacrity with which others switched to that solution. We are told that the movements of all dynamite wagons were traced and that the contents of all magazines checked, but that means nothing. Fischer's warning letters indicated a plot, but he is eliminated. The window weights seemed conclusive, but their presence may be explained, without a plot. No hole in the ground seemed to prove it was not dynamite, but those holding this theory overlook the fact that the explosive was several feet above the street with an air cushion below it to absorb the shock.

Colonel Dunn, whose opinion must be considered on all matters of explosives, believes, because of the character of the fragments, that it was not dynamite but a considerable quantity of a less powerful explosive in connection with gasoline. The regulations covering the use, transportation and storage of dynamite in New York City, embodied in Chapter 10 of the Code of Ordinances, provide adequate safeguards for our citizens when properly observed and strictly enforced. The Catskill Aqueduct from Yonkers to Brooklyn and all the new subways in congested city streets that used over a million pounds of dynamite yearly, were constructed without injury by dynamite to any one, other than the excavators.

The dynamite manufacturers and the responsible contractors mean to play safe, but much is left to subordinates. Magazines have been left unguarded, dynamite carried along our sidewalks and loaned and borrowed by contractors.

In one case children stole a box of dynamite from a magazine on Washington Heights, threw it over the cliffs and carried the scattered sticks home. A magazine in the Bronx, holding a thousand pounds of dynamite was found unguarded. Dynamite was taken from a magazine on Amsterdam Ave, while two policemen stood chatting on the curb. When things like that can happen it is not difficult to figure what may have happened on Wall St.

Wreckers have had permits to use dynamite to demolish buildings in congested New York streets. It is dangerous but has been done. Suppose they need dynamite and DuPont could not make an early delivery or they could not get a permit to blast? They would not get dynamite by the hundred pounds for they would not need that much and could not hide it from the inspectors. A few sticks, however, could be borrowed, placed in a wagon with a box of window weights removed from some other old building they were wrecking; add a couple of cans of gasoline for the air drill compressor and the detonators loose in the wagon. An auto hits the wagon, the window weights roll from the box, hits the exploders and detonate the dynamite, which explodes the gasoline.

Has anyone had experience enough with dynamite and gasoline exploding simultaneously to state as an expert, that there was no dynamite in this explosion? A bomb would simplify matters for all but the Reds, and it does not involve liability.

JOHN R. HEALY.

New York City Sept. 29.

[Mr. Healy was formerly on the engineering staff of the Catskill Aqueduct and later served as Inspector of Combustibles, Fire Department, New York City—EDITOR.]

Investigating Highway Bridge Impact

Sir—Impact in highway bridges, discussed in your editorial of Sept. 9, 1920, is in my opinion well worthy of investigation. I do not believe that the failure of late highway departments to make provision for impact is as serious a matter as the tone of your editorial appears to indicate; but present practice is not scientific and should be remedied.

As you state, tests have shown a very considerable impact on railroad bridges. Railroad bridges generally carry the full load for which they are designed, and sometimes they are overloaded. Therefore any increase in loading over the theoretical load is a serious matter. The trusses of highway bridges, however, are rarely called upon to carry anything like the loads for which they are designed, though the floor systems occasionally do carry loads approximating the assumed load. While there probably would be considerable impact under the load of a traction engine, this load would be only a small portion of the total capacity of the bridge, though it might be practically the full assumed load for the floor system.

If a study were made and the effect of impact produced by various highway loadings determined, it would be possible to design our bridges to meet conditions much more closely than is at present practiced, and I believe the necessary investigation should by all means be made.

M. W. TORKELSON,
Bridge Engineer, Wisconsin Highway Commission.
Madison, Wis., Sept. 21, 1920.

Waterproofing the Steelwork of Buildings To Prevent Corrosion

Sir—The interesting case of steel column corrosion described by E. H. Eardley on page 442 of your issue of Sept. 2, 1920, is a good illustration of the dangers attending the absence of waterproofing in metallic structures likely to be in contact with water and moisture. Engineers were not so alive to the importance of protecting metallic structures against corrosion at the time in which the Detroit Savings Bank Bldg. was built as they are now, though even at present proper attention is not always paid to the protection of steel in contact with damp foundations. The corrosion of the column in the Detroit Savings Bank Bldg. can be traced to contact with the seepage water used to wet the cinders, whose acidity caused local galvanic action.

Steelwork likely to be in contact with water ought to be carefully protected with suitable coatings of asphaltic materials, applying first a primer which penetrates the pores of the metal and insures the adhesion of the asphalt enamel applied in the molten state over the film of primer (or asphalt paint). Steelwork thus protected will withstand any causes of corrosion for a very long time.

The utmost care must be exercised in the application of the coatings of asphalt solution and enamel so as to avoid leaving even a minute space without protection. Imperfect and careless work will only increase cost without affording adequate protection. BAXERES DE ALZUGARAY.

New York City, Sept. 21, 1920.

HINTS FOR THE CONTRACTOR

Taking Cement Samples From Bags for Testing Purposes

BY S. WARREN
New York City

SAMPLING cement for testing is ordinarily a slow and dirty job. It is frequently done by untieing the bags and removing some of the cement from the top of the bag. The bags are usually carelessly retied, resulting in loss of cement in further handling. Sampling can be easily and quickly done by taking advantage of the fact that most cement bags are now filled through a valve in the bottom of the bag after it is tied. This valve is made by sewing a flap over a hole left in the stitching of one corner of the bag.

To get test samples secure a piece of thin brass tubing having a diameter of 1¼ in. and a length of 14 in. Stand the bag on the tied end, that is upside down. Insert the tube into the valve, then pull out the tube, holding the palm of the hand over the end of the tube. The contents of the tube can then be dumped into the sample bag.

TUBE IN PLACE FOR TAKING SAMPLE

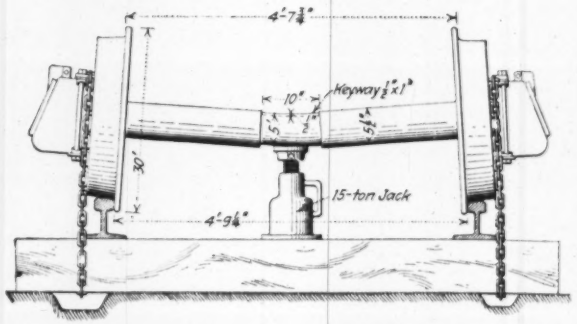
Bent Axle of Railroad Steam Shovel Straightened in Place

BY MORGAN CILLEY
Cumberland, Maryland

A BENT AXLE of a railroad steam shovel was straightened by a railroad yard crew in the field without removing the axle from the truck. Less labor was expended on the job than probably would have been necessary in putting in a new part and the cost was under $50. The shovel was a Marion type 1778, the property of the Vang Construction Co., and the work was done in the car yards of the Western Maryland R.R. near Cumberland.

The truck was run out from under the shovel, the axle spotted over a tie with the bend of the axle down. Holes were excavated under the tie beneath the journal boxes an dthe axle securely lashed to the tie with a 1-in. car chain passed over the journal boxes close to the wheel. The axle is 5½ in. in diameter except for a 10-in. length in the center which was turned down to 5 in. with a ½ in. x 1-in. key way cut into it. All the bend was in this portion and was sufficient to make a 1½-in. difference in the gage of the wheel, as shown by the cut. The cut also shows a general arrangement of the work together with various dimensions.

The bent portion was heated to an almost white heat with a crude oil burner, a boxing of sheet iron and wood having been built around the bent portion to confine the heat. A 15-ton jack was then placed under the axle by means of which the bend was straightened. It was a compound bend, two applications of the jack being necessary, but only one heating being required.



METHOD USED IN TAKING OUT BEND IN RAILROAD STEAM SHOVEL AXLE

The work was started at 2.30 p.m., the heat applied at 3.45 p.m., the pressure from the jack applied about 4.15 p.m., and the cooling started at 4.35 p.m. The truck was in place under the shovel at 5.30 p.m. Water was used to hasten the cooling and to temper the axle.

Accident to Temporary Railroad Trestle During Construction

THE necessity of careful inspection of temporary structures used during railway improvement work is illustrated by an accident which occured some time ago on the Chicago, Indianapolis & Louisville Ry. near Greencastle, Ind. To permit of rebuilding a culvert the track was supported on a pile trestle and the embankment was removed. The piles of the second bent were driven till they struck the footing of the old stone arch and as it was necessary to excavate the new foundation below that level these piles were cut off and supported on blocking built up of bridge stringers.

Before the concrete could be poured for the foundation there was a very heavy rain, and the end of the fill on the south side of the excavation became thoroughly saturated. After the water had subsided, the foreman examined the work but saw nothing which led him to anticipate trouble. A little later, however, the south end of the fill slipped and knocked down the bent of piles which had been supported on blocking. This slip occurred when a freight train was within a few hundred feet of the bridge and although an attempt was made to flag it, the train could not be stopped and the engine was derailed on the bridge. As the structure was short, the front truck was supported by the bank bent on the south side and the rear end of the engine was supported by the bent on the north side of the excavation. Damage to the engine amounted to less than $100, but it was necessary to detour traffic while the engine was being got back on the track and the trestle rebuilt.

NEWS OF THE WEEK

New York, October 7, 1920

Federal Highway Council Discusses Transportation

Plans Considered at Akron Meeting for Closer Co-ordination Among Several Interests Involved

A meeting of the Federal Highway Council, presided over by its chairman, S. M. Williams, was held at the Portage Hotel and City Club, Akron, Ohio, Sept. 27 and 28.

Approximately 150 members representing all classes interested in the development of the nation's highways were present and took active part in the conduct of the meeting. Evidence of a growing realization of interdependence of steam and electric railways and waterways with the highway development of the country was shown by the attendance of M. S. Conners, general manager of the Hocking Valley Railroad, representing the American Railroad Association; S. S. Holbrook, vice-president and treasurer of the American Railway Express Company; G. T. Seeley, vice-president and general manager of the Penn-Ohio Electric Co., representing the American Electric Railway Association, and Dr. R. S. MacElwee, director of the Bureau of Foreign and Domestic Commerce, Department of Commerce, Washington.

Committees were formed to study the country-wide situation with a view to making future recommendations looking toward closer co-ordination and co-operation among the various transportation interests.

Recognizing the fact that one of the primary considerations in the development of the highway system of the country is the betterment of foundation and sub-grade conditions, the committee on sub-grade and its relation to road surface and traffic made a progress report covering the thorough and widespread investigation of all factors entering into the study of sub-foundations and their relations to the integrity of the entire highway structure.

A progress report was rendered by W. J. L. Banham, chairman of the transportation committee, on the relations of highway to railroads and waterways, the object of which is to effect co-operation with railways, waterways and other forms of transportation. Sub-committees of the transportation committee reported on studies of short haul and terminal problems.

The rural motor express sub-committee reported upon studies of this committee under the general transportation committee, looking toward more

Am. Soc. C. E. Vote on Amendments Canvassed

Result Announced Last Night at Meeting of the Society in New York — Seven Proposed Changes in Constitution Involved

At the meeting of the American Society of Civil Engineers held last night in New York City the vote on the seven proposed amendments to the constitution, which have been under consideration since a letter ballot was issued to the corporate membership Aug. 28, was canvassed. By delaying the printing of its last form *Engineering News-Record* is able in this issue to present the results of the referendum. Lack of time, however, prevented the inclusion of such details as the number of votes "for" and "against" each question or an analysis of the vote by districts. It is planned to publish all of this information in detail next week.

The passage of an amendment to the constitution requires a two-thirds majority of the votes cast. The outcome of the balloting was as follows:

Amendment A—Enlarging scope of society to include co-operation in economic and civic affairs.
Defeated

Amendment B—Establishing Local Sections and assigning all members thereto.
Defeated

Amendment C—Increasing annual dues.
Carried

Amendment D — Election of Honorary Members.
Carried

Amendment E—Forbidding members of nominating Committee to be candidates for office.
Carried

Amendment F—Reducing number of directors from the New York District.
Defeated

Amendment G—Making changes in the method of nominating and electing officers and providing for conference of Local Section representatives.
Defeated

extensive adoption of rural motor express service.

David Beecroft, directing editor of the Class Journal Publishing Co. and president of the American Society of Automotive Engineers, reported the activities of the committee for study of highway transport functions of state highway departments, including traffic surveys, traffic control and snow removal from highways. Recommendations by this committee to the Federal Highway Council covered forwarding of questionnaires to the highway departments of the several states to learn what each is doing in connection with conducting traffic census and highway transport surveys, looking toward development of a uniform system of gathering and tabulating of highway traffic data with a view to applying information thus gained to the construction of future highways.

RECOMMEND TRANSPORT ENGINEER

Recommendation was made to the Council that effort be made to secure the creation of a highway transport engineer in each of the state highway departments, whose duties would be the making of highway transport surveys for the purpose of co-ordinating all factors entering into future development of the nation's highway service.

Prof. A. H. Blanchard of the University of Michigan reported activities of his committee toward the co-operation of universities and schools in the development of highway engineers and for the general dissemination of accurate information on highway subjects.

W. A. Allsdorf, Columbus, Ohio, chairman of the committee on international highway development, submitted a progress report covering highway activities in foreign countries, particularly stressing the fact that the Federal Highway Council, through this committee, is constantly receiving from foreign countries requests for assistance and advice based upon experience in the United States.

Dr. R. S. MacElwee, chairman of sub-committee on store-door delivery laid emphasis upon the necessity for the perfection of the store-door delivery movement as a means for reducing congestion at railroad and steamship terminal points.

During the session there was presented a film prepared by the Federal Highway Council, entitled "Virginia's New Hour," depicting the existing conditions in Virginia. This film is now being shown throughout the state of Virginia, awakening a tremendous interest in the development of its highways to a position more nearly commensurate with the development of the industrial and agricultural needs of the state.

The members of the Federal Highway Council attending the conference were guests of the Akron Chamber of Commerce at the Portage Country Club, Sept. 27, at which time addresses were made by M. S. Conners, general manager of the Hocking Valley Railroad, on the need for greater co-operation between highway interests and the railroads and S. F. Taber, president of the Ohio State Grange, on the urgent need of better highways in the rural districts for the transportation of farm products to the market.

Engineers in Relation to Port and Harbor Design

The close relations of engineering to port development along the varied lines of general economic planning, design of structures and handling of materials were brought out prominently during the convention of the American Association of Port Authorities, held in Chicago, Sept. 30 to Oct. 2. Local facilities, conditions, rates and commerce at several individual seaports were presented in one group of papers. Inland navigation and lake ports were the subject of another group.

Belt railways at ports should be owned preferably by the public, according to W. O. Hudson, New Orleans, and from his experience he preferred state ownership to city ownership, as being less affected by political influences. In regard to new ports, he pointed out that facilities alone will not bring ships, but ships will come if a port can offer cargo. When the discussion drifted into the technical features of the work at New Orleans, Mr. Hudson suggested that the meeting ought to have more papers on constructive subjects by engineers. He referred also to the new industrial canal as a step toward a new outlet to the Gulf independent of the Mississippi River. Belt railway service at San Francisco, Seattle and Montreal was touched upon. Warehouses convenient to piers and railways were advocated by several speakers in order that freight might be unloaded from cars promptly instead of having the cars held in storage yards until a ship is ready for cargo.

WIDE PIERS PREFERRED

Wide piers or quays with transit sheds, tracks and warehouses were considered preferable to narrow piers by R. S. MacElwee, U. S. Bureau of Foreign and Domestic Commerce. The author showed views and plans of numerous ports and port facilities. The provision of tracks along the sides of piers was discussed at some length, one opinion being that for a length of over 600 ft. there should be two tracks, so as to permit movement past cars spotted at vessels. One speaker referred to labor influences in preventing economical handling of freight, but others thought that labor must and will fall in line with improvements. As to general design of piers, it was suggested that economy and efficiency of service for general use should be the basis. In too many cases piers have been designed to suit some particular business or lessee; then when the business or lessee changes the pier is found unsatisfactory and blame is laid upon its design.

The "practical man" is superior to the engineer in port design, according to George F. Sproule, Director of Wharves, Docks and Ferries of Philadelphia, who attributed the secondary maritime position which the United States occupies to "the fact that the development of her ports and harbors has been entrusted to impractical and technical men." He said, too, that "the usefulness of the engineers lies largely in their carrying out scientifically the fundamental ideas of the practical man." Issue was taken promptly with his statements, and it was advanced not only that the engineer is the logical man to design port development works, acting in co-operation with commercial interests, but that unsatisfactory conditions have resulted where the "practical" man has insisted on having his ideas carried out.

CONSTRUCTION METHODS DISCUSSED

A monolithic reinforced-concrete construction for docks or bulkhead walls, proposed by John Ericson, Chicago, consists of interlocking face piles or grooved face piles with intermediate precast panels which are grouted in place. There was some discussion as to the practicability of keeping piles in line. Casting the panels in place behind sheeting driven outside of the face piles was suggested as a simpler method by F. M. Williams, State Engineer of New York. Mr. Ericson estimated the cost of his system at $57 to $86 per lineal foot, as against $79.50 for a timber bulkhead, $106 for timber with concrete coping and $292 for a gravity concrete wall.

Inland navigation was dealt with in papers on the Manchester ship canal, the New York State barge canal, the Illinois River waterway and some lake port developments. These latter included the Toronto harbor works, a rail and water terminal project for Detroit and the Calumet harbor project at Chicago to eliminate the Chicago River as a hindrance to both land and water transportation.

Amendments to the constitution were adopted which provide, among other matters, for a larger representation of engineers. Benjamin Thompson, of Tampa, Fla., is the new president; M. P. Fennell, Jr., Montreal, Can., is secretary. The next meeting is to be held at Seattle, Wash., in 1921.

Q. M. Construction Projects

The principal projects engaging the attention of the construction division of the Quartermaster Corps, U. S. A., at the present time are the following: Machine shop, Fort Worden; hangar, Aberdeen Proving Ground; Quartermaster shops, Fort Meyer; ingot yard, Watertown Arsenal; stables, Fort Wadsworth; machine shop, Fort Adams.

Progress Made in Standardizing Construction Accident Records

Definite progress in standardizing records of construction accidents was made by the Construction Section of the National Safety Council at its Annual Congress held last week at Milwaukee, Wis. Two reports on the subject were presented by the Statistical and Standardization Committee consisting of F. A. Davidson, Dwight P. Robinson & Co., Inc.; F. S. Robinson, General Builders' Association, Detroit; and J. E. Griffith, E. I. Du Pont De Nemours & Co. One report recommending a method of determining and recording rates of accident frequency and accident severity was accepted and the method recommended was adopted as standard. The second report on classification of accidents according to causes, was, after discussion, referred back to the committee for revision. The objection raised to the second report was that it failed to conform in classification to the practice of the National Safety Council and the U. S. Bureau of Labor. Instruction was given the committee that it conform its classification to that generally adopted with a sub-classification of causes peculiar to, or particularly prevalent in, construction operations.

In the discussion of the reports, stress was laid on the facts that until uniform methods of accident determination and recording were practised, constructors could not directly compare their statistics nor bring statistics in a compelling form to influence the rates of liability insurance companies. At present these rates for construction work are based on partial and irreconcilable data and constructors have nothing better to offer in urging rate revision, either for the construction industry as a whole or for the less hazardous kinds of construction.

Adoption of accident prevention methods by the small contractor was the subject of active discussion following the paper by O. H. Ulbricht, secretary, Master Builders' Association of Wisconsin. In referring to the subject stated, Mr. Ulbricht said: "We feel that the Master Builders' Association of Wisconsin has to a very large extent solved the problem of how to interest the contractor in safety. Our association consists of over 1,100 members, who are located in over 100 cities of Wisconsin. We issue a monthly 12-page publication, *The Builders' Bulletin*, devoted to the interest of the members and to the building trade in general, also to safety propaganda, but our crowning achievement, one which has accomplished more than anything else for the contractors of the state in educating them to observe safety measures in all of their undertakings, is our Builders' Insurance Co. The loss ratio of this company, as well as its premium rates, are the lowest of any similar company in Wisconsin. It writes compensation and automobile liability insurance for builders only, who are members of the state association and in addition to preaching and spreading safety and accident prevention information constantly, undoubtedly thereby saving many a life and limb, it has saved its members thousands of dollars by providing for them insurance at cost."

Without developing formal action the conclusion reached as the result of the presentation by a number of speakers of the limitations and difficulties of the small contractor in practicing scientific accident prevention was: Accident prevention work by the small contractor must be a group enterprise; local contractors' organizations acting for all their members and charging these members pro rata must organize accident prevention work either in conjunction with mutual liability insurance or separately as purely an accident prevention measure.

To Form Cornell Engineering Society

At the annual meeting of the Cornell Society of Civil Engineers, to be held at the Cornell University Club, New York City, Oct. 19, 1920, amendments to the constitution are to be voted upon which will change the name of the Society to the Cornell Society of Engineers. This move is a result of the joining of all the engineering colleges at Cornell under one head. Hitherto only graduates of the civil engineering college have been eligible for membership in the society and there has been no other engineering society. It is hoped that the new association will attract a large number of the graduates of Sibley College.

At the annual meeting the new dean of the combined engineering schools, Dexter S. Kimball, will deliver an address, and Clifford M. Holland, chief engineer of the New York State Bridge and Tunnel Commission, will speak on the new Hudson River highway tunnel.

Glasgow Prepares for Engineering and Shipbuilding Exposition

The Glasgow Corporation, according to a recent issue of *Commerce Reports*, is making preparations for the engineering, shipbuilding, and electrical exhibition that is to be held in the Kelvin Hall, Glasgow, from Nov. 8 to Dec. 4. This hall has an area of over 200,000 sq.ft. and is well adapted for such an exhibition, and will contain, when the show is open, 3 mi. of passageways with stands on both sides. A moving picture hall is being planned in connection with the exhibition to show films devoted to the engineering, shipbuilding, and electrical industries. The exhibitors are to include most of the well-known British engineering firms. It is reported that over 15,000 personal invitations have been dispatched to foreign visitors, for whose benefit, during their stay in Glasgow, advisory bureaus are to be set up by the corporation at the principal Glasgow stations.

New Plan for Solution of Imperial Valley Problems

Both flood control and increased water supply for additional areas to be brought under cultivation in Imperial Valley, Cal., are provided for in a plan contemplating the storage of Colorado River flood waters in Volcano Lake which was announced by J. C. Allison at a recent meeting of the Los Angeles Section, American Society of Civil Engineers. The plan represents the results of several years of investigation conducted by Mr. Allison in conjunction with other engineers. The work, he stated, would cost only one-third to one-fifth as much as the All-American canal scheme, which he declared to be impracticable.

The new plan provides for the creation of a storage reservoir in Volcano Lake to impound about 1,200,000 acre-ft. of water. This supply could be used to irrigate 500,000 acres of additional land in Imperial Valley, most of which lies on the American side of the international boundary. Surveys for the necessary high line canals have shown that these can be so located that the cost of the entire project will be less than $10,000,000. Under this plan the Colorado River would be returned immediately to its old channel. One result of the construction of the Volcano Lake reservoir, Mr. Allison stated, would be a high bank of silt 25 miles long that would constitute a permanent barrier between Imperial Valley and the main river channel thereby affording protection against the annual flood menace.

Attack on Pittsburgh Steel Price Basing to Get Rehearing

It is reported that the Federal Trade Commission has decided to give a rehearing to the complaint recently brought before it by the Western Association of Rolled Steel Consumers, attacking the practice of basing steel prices on Pittsburgh regardless of where the steel is produced. This case on its recent hearing was decided against the complainant, and the practice of fixing prices by adding the freight charge to the Pittsburgh base no matter where the steel is made was approved. The rehearing is to begin Nov. 15, 1920.

To Manage Toronto's Street Railways

In preparation for the taking over of the street railways by the City of Toronto, Ont., the Transportation Commission of the city has appointed a board of consulting engineers and a general manager. The board consists of R. C. Harris, commissioner of works of Toronto; E. L. Cousins, general manager of the Toronto Harbor Commission, and F. A. Gaby, chief engineer of the Hydro-Electric Power Commission of Ontario. H. G. Couzens, manager of the Toronto Hydro-Electric System, has been made general manager of the new city transit work.

Military Affairs Committee of Engineering Council Reports

Urging greater co-operation among military and civilian engineers through the medium of temporary interchanges of duties, the Military Affairs Committee of Engineering Council has recently submitted a progress report signed by Col. William Barclay Parsons, chairman, and Col. George D. Snyder, secretary. The other members of the committee are Paul G. Brown, H. W. Buck, George Gibbs, Spencer Miller, Dr. Leonard Waldo and Cols. R. D. Black, F. J. Miller, F. A. Snyder, F. A. Molitor, B. W. Dunn, J. J. Carty, Captain (Navy) A. B. Fry, Lt.-Col. F. E. Humphreys and Major A. S. Dwight. After emphasizing the importance of military training for the civilian engineer and the features of the Officers Reserve Corps the report suggests that regular army officers should receive temporary assignments to civilian engineering and construction organizations. The report, in part, follows:

But the bringing together of the two great divisions of engineering science, military and civil, is only partly done when there is extended to civilian engineers the opportunity for gaining experience as military engineers. The full result will be attained when the military engineers, in a corresponding manner, are given the opportunity to become experienced as civil engineers. To enable the members of the Corps of Engineers to gain experience in all branches of civil engineering, they should have opportunities for service on as much as possible of the general construction work done by the Federal Government, and not be confined to river and harbor improvements, as has been the custom. In addition to this members of the Corps should be assigned for temporary employment with state highway commissions or private corporations such as railway companies, metallurgical establishments, large contracting firms or factories where machinery, electrical devices and other forms of manufactured articles, in which engineers are interested, are produced and where members of the Corps can become acquainted with all phases of practical operation.

Not even the best technical schools, where the course of four years follows a previous collegiate education of at least two years, can do more than give the students the rudiments of theoretical training. How much less, therefore, can the Military Academy, where the larger part of the course is necessarily devoted to subjects other than engineering, turn out broadly trained engineers with practical experience?

The War Department has established line and staff schools where officers can take post-graduate courses in military subjects. The opportunity herein suggested would provide the corresponding post-graduate courses for engineers and on the soundest basis.

Your committee believe that arrangements can be made without difficulty whereby officers of the technical services would be received as members of the operating staffs of transportation companies and engineering corporations. The advantage in thus broadening the experience of army engineers in fields of which they must know much in times of war are obvious. If while they are temporarily withdrawn from military duties there are insufficient engineers to meet the requirements of the War Department, the advantage would be presented to give junior officers the experience of exercising judgment and assuming authority at an age much earlier than would otherwise be the case, while the temporary vacancies in lower grades could be filled by assignments from the Reserve. Should some officers, after trying work in civil life, decide to remain in the new field of employment the army and the country would be the gainers. Permanent promotion would be accelerated, a most desirable thing in time of peace, and there would be added to the Reserve, ready for call in case of war, engineer officers well trained in military knowledge.

We therefore recommend that the question of the proper utilization of engineer officers and the means to adopt whereby the best type of engineers from civil life can be attracted, be taken up with the chiefs of the technical services where the power of initiative lies. The present moment appears to be highly propitious. For nearly two years engineers from civil life, some holding temporary commissions as officers of the Army, others still retaining civil status, have worked with the officers of the Corps of Engineers and other technical services, both in the United States and in the field in France. There exists as never before a mutual respect between the officers of the technical corps of the Army and the great body of professional engineers based on close intimacy and common service. That mutual feeling possesses a great force of latent momentum. Before it is dissipated by time it should be intelligently directed for the better service of the country.

To Discuss Merits of Steam and Electric Locomotives

A joint meeting to discuss the relative merits of modern steam and electric locomotives will be held by the New York Section of the Am. Inst. E. E., the Metropolitan Section of the Am. Soc. M. E. and the Railroad Section of the Am. Soc. M. E., Oct. 22, 1920, at the Engineering Societies Building, 29 West 39th St., New York City. Papers will be presented by J. R. Muhlfeld, Railway and Industrial Engineers, Inc.; W. E. Woodard, vice-president, Lima Locomotive Works; A. H. Armstrong, chairman, electrification committee, General Electric Co. and F. H. Shepard, director of heavy traction, Westinghouse Electric & Manufacturing Co.

Virginia Board of Examiners Is Appointed

The appointment of members of the Virginia State Board for the examination and certification of architects, professional engineers and land surveyors has been made by the governor. The law was approved on March 19. The appointments are as follows: P. B. Winfree, engineer, Lynchburg; James F. MacTier, engineer, Roanoke; John Kevan Peebles, architect, Norfolk; Fiske Kimball, architect, University of Virginia; W. C. Noland, architect, Richmond; Thomas M. Fendall, surveyor, Leesburg; L. B. Dutrow, surveyor, Peyersburg; W. D. Tyler, engineer, Dante; C. G. Massie, Amherst. At an organization meeting Sept. 17, Mr. Kimball was elected president and Mr. Dutrow, secretary *pro tem.*

Illinois Society Offers Prizes for Papers

To stimulate interest among its younger members the Illinois Society of Engineers has opened a competition for technical papers in the five divisions of drainage, roads and pavements, sewerage, surveying and miscellaneous civil engineering. A prize of $25 and a certificate of merit will be awarded to the best paper in each division, the papers to be from 1,500 to 2,500 words in length. This competition is limited to authors who have been members of the society for not more than five years on Jan. 1, 1921.

Consulting Engineers Condemn City for Unprofessional Practice

An advertisment appeared in North Tonawanda, N. Y., and presumably in other cities in the western part of New York State, reading as follows:

NOTICE TO ENGINEERS

Sealed proposals, endorsed "Proposal for Sewerage and Drainage Survey and Report" and addressed to Fred C. Goltz, City Clerk of the City of North Tonawanda, N. Y., will be received and opened at the office of the Board of Public Works at 8 p.m. on the (12th) twelfth day of April, 1920, in accordance with the specifications on file in the office of the City Clerk. All bids must be accompanied by a certified check or bid bond to the City of North Tonawanda, N. Y., in the sum of ten per cent (10%) of the total amount of the bid, as a guarantee of good faith. An additional Surety Bond of 50% of the contract, must be furnished by the successful bidder. The Board of Public Works reserves the right to reject any and all bids if they deem it for the best interest of the City to do so. Specifications can be seen and proposals obtained from the City Engineer.

BOARD OF PUBLIC WORKS.
By CARL L. OELKERS,
City Engineer.

This matter was taken up by the Committee on Relations of the American Institute of Consulting Engineers, which made a report on the subject and recommended the following resolutions which were adopted by Council at its meeting Sept. 15:

WHEREAS, it has come to the attention of the Council of the American Institute of Consulting Engineers that the City of North Tonawanda advertised in March and April for sealed bids to be sent to the City Clerk of that City for undertaking the professional engineering work of a sewerage and drainage survey and report, the said bids to be accompanied by a certified check or bid bond in the sum of 10 per cent of the proposed contract price, and an additional surety bond for 50 per cent of the contract price to be furnished by the successful bidder; and

WHEREAS, the conducting of such surveys and the designing of sewerage systems is professional engineering, and it is important to the welfare of the public that it shall be dealt with only in professional manner by experienced and capable sanitary engineers; and

WHEREAS, the procedure of the City of Tonawanda is detrimental to the public interest in that (1) it subordinates the professional aspects of this engineering work to commercial considerations, and that (2) professional engineers of the best standing cannot enter upon work in the competitive manner herewith proposed; therefore

RESOLVED, that the method of arranging for engineers for its sewerage and drainage survey and report utilized by the City of North Tonawanda is contrary to the public interest and must be condemned by the engineering profession; and

RESOLVED, that a copy of these resolutions shall be sent to the City Engineer and City Clerk of the said city and that the resolutions also be given general publicity.

Five San Francisco Contractors Indicted by Grand Jury

Five San Francisco contractors were last week indicated by the grand jury as being involved in the "inner ring," or contractors' trust, accused of combining to raise bids on grading work in that city. The five contractors named are William S. Scott, William H. Healy, Frank Mordecai, Frank O'Shea and R. A. Farrar. A statement of the charges of conspiracy appeared in *Engineering News-Record*, Sept. 23, p. 826. Investigations are to be continued to expose the entire system.

A. S. M. E. Names Representatives for American Engineering Council

The Council of the American Society of Mechanical Engineers has appointed the society's representatives on the American Engineering Council, the governing body of the newly created Federated American Engineering Societies. The delegates appointed will hold office for two years beginning Jan. 1, 1921, and will also represent the society at the organization meeting of the American Engineering Council to be held at Washington, D. C., Nov. 18 and 19. The list of representatives appointed follows:

L. P. Alford, formerly editor *Industrial Management;* E. S. Carman, secretary and chief engineer, the Osborn Manufacturing Company, Cleveland; R. H. Fernald, professor mechanical engineering, University of Pennsylvania, Philadelphia; A. M. Greene, Jr., professor mechanical engineering, Rensselaer Polytechinc Institute, Troy, N. Y.; W. B. Gregory, professor experimental engineering, Tulane University, and irrigation engineer, U. S. Department Agriculture, New Orleans; W. A. Hanley, master mechanic and chief engineer, Eli Lilly & Co., Philadelphia; D. S. Kimball, professor industrial engineering, Cornell University, Ithaca, N. Y.; Charles T. Main, consulting engineer, Boston, Mass.; Fred J. Miller, president of A. S. M. E., 1920; L. C. Nordmeyer, secretary and treasurer, Tait and Nordmeyer Engineering Company, St. Louis; V. M. Palmer, engineer of industrial economy, Eastman Kodak Co., Rochester, N. Y.

ENGINEERING SOCIETIES

Calendar

Annual Meetings

AMERICAN SOCIETY FOR MUNICIPAL IMPROVEMENTS, Valparaiso, Ind.; St. Louis, Oct. 12-15.
AMERICAN RAILWAY BRIDGE & BUILDING ASSOCIATION, Chicago; Atlanta, Ga., Oct. 26-28.
NATIONAL DRAINAGE CONGRESS, Chicago; Atlanta, Ga., Nov. 10-12.

The Association of Professional Engineers, Province of New Brunswick, held its first meeting, after the passing of the act regarding registration of engineers by the Provincial Legislature last spring, at St. John, Sept. 10. By-laws were adopted and the following officers and members of council were elected: President, C. C. Kirby, St. John; vice-president, C. B. Brown, Moncton; councillors, St. John District, J. A. Grant, 1922, F. P. Vaughan, 1921; Moncton District, S. B. Wass, 1923, J. Edington, 1921; Fredericton District, B. M. Hill, 1923; Chatham District, Geoffrey Stead, 1922. A nominating committee was elected to draw up a slate for the annual meeting next January, and A. R. Crookshank was appointed secretary-treasurer and registrar.

The Western Society of Engineers has listed the following subjects for October: Oct. 11, "Bascule Bridges at Home and Abroad," by Philip L. Kaufman, Strauss Bascule Bridge Co.; Oct. 18, "Development of the Illinois River for Navigation," by M. G. Barnes, chief engineer of the State Division of Waterways. On Oct. 20 there will be a joint meeting with the Chicago Section of the American Institute of Electrical Engineers for the annual lecture by Dr. Charles P. Steinmetz. On Oct. 21 there will be a joint meeting with the Chicago Section of the Railway Signal Association, with a paper on "Automatic Train Control" by W. P. Borland, Interstate Commerce Commission, and W. B. Murray, engineer of the Miller Train Control Co.

The City Managers' Association will hold its seventh convention Nov. 15-17 at Hotel Sinton, Cincinnati, Ohio.

Candidates for Am. Soc. C. E. Offices Are Named

The American Society of Civil Engineers has just announced the Nominating Committee's selection of candidates for offices to be filled at the annual election Jan. 19, 1921. The ticket follows:

For President: George S. Webster, Philadelphia.

For Vice-presidents: Andrew M. Hunt, New York, and Edward E. Wall, St. Louis.

For Treasurer: Otis E. Hovey, New York.

For Directors: John P. Hogan and Ira W. McConnell, New York (Dist. 1); Richard L. Humphrey, Philadelphia (Dist. 4); Baxter L. Brown, St. Louis (Dist. 9); Frank T. Darrow, Lincoln, Neb. (Dist. 10); George G. Anderson, Los Angeles (Dist. 11).

PERSONAL NOTES

JOHN GREER has been appointed assistant engineer of the Portland Division, Grand Trunk Ry., with headquarters at Portland, Me. He succeeds R. F. Nicholson, resigned.

CARL C. THOMAS, consulting engineer, Los Angeles, Cal., has been appointed Western representative of Dwight P. Robinson & Co., New York, with offices in the Electric Equipment Building, Los Angeles.

WILFRID S. LAWSON, Ottawa, Ont., has been appointed bridge and structural engineer for the Department of Railways and Canals, Ottawa.

MAJOR E. N. JOHNSTON, Corps of Engineers, U. S. A., in charge of work in the Wilmington District, has been directed to take over, in addition, the direction of the work of the Baltimore District. The recent retirement of Colonel J. P. Jervey left the latter district without a head and, due to the shortage of engineer officers, it has been necessary to place double responsibilities on Major Johnston.

J. W. FOX, valuation engineer of the Central of Georgia Railway Co., and chief engineer for the corporation during Federal control, has resigned to become associated with the Lawrence Construction Co., general contractors, Augusta, Ga.

BANKS & CRAIG is the name of a new firm of consulting engineers and chemists established by Robert Hall Craig, formerly sanitary engineer with the Surgeon General of the Army and more recently sanitary and hydraulic engineer with the Construction Division of the Army, and Henry Ward Banks, 3d, formerly research chemist with the Harriman Laboratory, New York, and the National Biscuit Co. The offices of the firm are at 134 E. 44th St., New York.

EISENHARDT-CONKEY CO., Evansville, Ind., is a new firm organized by Karl J. Eisenhardt, recently division engineer of the Indiana State Highway Commission, at Terre Haute, and V. P. Conkey, civil engineer, Evansville. The firm will specialize in highway, drainage and mining engineering.

GEORGE L. BURTON, of South River, N. J., has been appointed a member of the New Jersey State Highway Commission. He was nominated by the Governor as a member of the commission Sept. 8, with seven others. The Senate then confirmed all of the nominations except that of Mr. Burton, who was rejected on the ground that he had patented paving connections.

L. O. BERNHAGEN, who has been connected with the Texas State Board of Health as sanitary engineer, at Austin, has resigned to accept a similar position with the City of Beaumont.

S. S. GANNETT has completed his duties as chairman of the Arkansas-Mississippi Boundary Commission, to which he was appointed by the U. S. Supreme Court, and resumed his work as a topographer with the U. S. Geological Survey.

C. C. HOLDER, of the Topographic Section of the U. S. Geological Survey, has been sent to Houston, Tex., to assist in the revision of the map of that city.

IRA O. BAKER, professor of civil engineering, University of Illinois, has been acting head of the civil engineering department of the university, succeeding Prof. Frederick H. Newell, recently resigned.

ARTHUR CUTTS WILLARD, professor of heating and ventilation, department of mechanical engineering, University of Illinois, has been appointed head of the department to succeed Dean C. R. Richards, who, since his election as dean of the college of engineering and director of the Engi-

neering Experiment Station, has continued to serve as the acting head of the department of mechanical engineering.

LT.-COL. JAMES FRANKLIN BELL, Corps of Engineers, U. S. A., at New Orleans, has been appointed to take charge of the Pittsburgh District. The office has been vacant since Colonel H. B. Ferguson left for Washington. Colonel C. W. Otwell, of the Wheeling District, has been temporarily assigned to this district.

HICKS, CRADDOCK & MCKAY is a new firm of engineers who will conduct a general engineering business in Centralia, Wash. The members of the firm are C. P. Hicks, who has had twelve years' experience in logging, railway location and general engineering; F. Harper Craddock, until recently located in Oklahoma engaged in municipal engineering and appraisals, and Neil R. McKay, recently in the city engineer's office at Olympia.

JOSHUA B. F. BREED, former city engineer and for a number of years chief engineer for the old sewer commission of Louisville, Ky., has been appointed chief engineer of the new sewer commission of that city.

W. C. ALLEN has been appointed assistant valuation engineer of the Fort Worth and Denver City Railway Co. and the Wichita Valley Railway Co.

OBITUARY

SAMUEL M. SOUDER, formerly superintendent of the R. D. Wood Iron Works, of Millville, N. J., died in that town Sept. 27. During the Civil War Mr. Souder was engaged in shipbuilding at Cramp's shipyards. Later he constructed the water-works at Manchester, N. H., and a similar plant at Lynchburg, Va. He was also in charge of the building of the dam at Millville. Mr. Souder was born at Millville in 1837.

RICHARD L. O'DONNEL, vice-president of the Pennsylvania Railroad Co., at Pittsburgh, died, Sept. 28, at New York. He was born in New York Nov. 5, 1860, and received his civil engineering education in the Polytechnic College of Pennsylvania. In 1883 he entered the service of the Pennsylvania Railroad Co. as civil engineer; three years later he became assistant engineer in the principal assistant engineer's office at Altoona and later was stationed at Hollidaysburg and Lancaster. Mr. O'Donnel was on duty at Johnstown after the flood in 1889, and after serving in various capacities at other points in Pennsylvania he was transferred to the Pittsburgh division as assistant engineer in 1894. After several further promotions, he was made assistant general manager of the lines east of Pittsburgh, in 1917, with a promotion the following year to the position of general manager, and on March 1 of last year he was made vice-president of the central region with offices at Pittsburgh. When our troops were on the Mexican border Mr. O'Donnel was general agent of the American Railway Association at Governors Island, and served in the same capacity during the World War. He had jurisdiction over the rail transportation of troops from the various states to the border, and later between the camps and embarkation piers in the movement overseas.

WILLIAM HENRY DIETRICH, resident engineer at Bombay, India, of the U. S. Steel Products Co., died in that city Aug. 11. He was born near Waynesboro, Pa., in 1873. He attended the Mercersburg (Pa.) Academy and later graduated in civil engineering from Princeton University, in which he afterward was an instructor in the department of civil engineering. The greater part of Mr. Dietrich's professional career was spent with the American Bridge Co. and the U. S. Steel Products Co. In the early part of his connection with the latter company he had charge of erection of bridges in Peru. Later he was sent by it to Shanghai, China, in the capacity of resident engineer and in January of this year was transferred to Bombay.

GEORGE W. COOLEY, for more than ten years state engineer and secretary of the Minnesota Highway Commission, died in Minneapolis, Sept. 25. Mr. Cooley was one of the pioneer highway engineers of the state, having laid out the present road system.

DR. ADOLPH GEHRMANN, professor of bacteriology and hygiene, College of Physicians and Surgeons, University of Illinois, and founder of the Columbus Laboratories, died, Oct. 3, at Chicago.

BUSINESS NOTES

A. G. HARKINS, for fourteen years associated with the sales department of the Carnegie Steel Co., the last eight years having been in charge of its Baltimore sales offices, has resigned to assume an active partnership in the Maryland Steel Products Co., Calvert Building, Baltimore. During the war Mr. Harkins was connected with the American Iron and Steel Institute's Washington office, handling the distribution of various steel products for the Government.

WHITING CORPORATION is the name of the new organization formed by consolidation of the Whiting Foundry Equipment Co., Harvey, Ill., and the American Foundry Equipment Co., New York. J. H. Whiting, president of Whiting Foundry, becomes chairman of the board, and V. E. Minich, president of American Foundry, will be president. It is the intention to continue and enlarge the present offices of the latter concern in New York as the Eastern sales and export office of the combined lines.

THE SANDERSON-CYCLONE DRILL CO., Orrville, Ohio, has appointed W. F. Nothacker as vice-president in charge of its Eastern and export office, 30 Church St., New York.

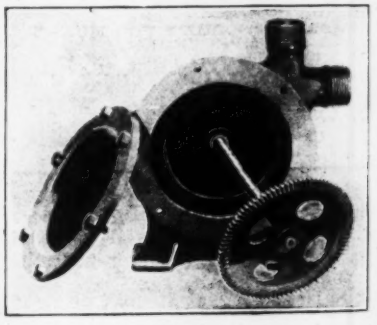
J. CHANDLER SMITH, for the past six years associated with the sales department of the Carnegie Steel Co. as assistant manager of its Baltimore sales offices, has resigned to assume an active partnership in the Maryland Steel Products Co., Calvert Building, Baltimore.

R. H. BEAUMONT CO., of Philadelphia, contractor for conveying and hoisting systems and complete power plants, has recently opened an office at 230 Fifth Ave., Pittsburgh, Pa., with Thomas Widdop in charge.

CONVEYORS CORPORATION OF AMERICA is the new corporate name of the American Steam Conveyor Corporation, Chicago. The change in name is due to the addition by the company of new lines of business and the purchase at South Bend, Ind., of a large machine shop. The new name more nearly describes the present business of the organization, its lines including steam jet conveyors for handling ashes, soot, combustion ash, etc.; trolley carriers for handling coal, limestone, sand, gravel and like loose bulky materials; ash pit doors; flood oil bearing sheaves and other conveyor supplies.

Pump With Disk Impeller

A centrifugal pump which has both the intake and discharge openings on the periphery and in which the revolving element is a disk having one or more blades or projections on its circumference is being introduced by the Western Pump Co., Davenport, Iowa.



DISK IMPELLER PUMP

under the name of the Westco pump. The disk or impeller revolves at high velocity, drawing in water through the intake to fill the chambers between the projections and discharging it through the adjacent outlet. This pump, shown in the accompanying illustration, is made in various sizes up to a capacity of 60,000 gal. per hour.